THREE SPAN ARCH SUBWAY
OF THE
C. M. & St. P. Ry. OVER HUMBOLDT BOUL. CHICAGO

BY

P. F. GREIFENHAGEN
R. J. KOCH
H. S. WALIN

ARMOUR INSTITUTE OF TECHNOLOGY

1913

625.4
G 86

Illinois Institute
of Technology
Libraries

AT 296
Greifenhagen, P. F.
Proposed three span arch
subway of the C. M. & ST.

PROPOSED THREE SPAN ARCH
SUBWAY OF THE C. M. & ST. P. RY.
over
HUMBOLDT BOULEVARD.

* *

A THESIS PRESENTED BY

AND

Raymond J. Koch

AND

TO THE

PRESIDENT AND FACULTY

OF

ARMOUR INSTITUTE OF TECHNOLOGY

FOR THE DEGREE

OF

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

HAVING COMPLETED THE PRESCRIBED COURSE OF STUDY

IN

CIVIL ENGINEERING,

H M Raymond

L. C. Monin

ILLINOIS INSTITUTE OF TECHNOLOGY
PAUL V. GALVIN LIBRARY
35 WEST 33RD STREET
CHICAGO, IL 60616

625.4
G86

ACKNOWLEDGMENT.

We wish to acknowledge the great assistance rendered us by Assistant Professor of Civil Engineering, Stanley Dean. We received valuable information from the Engineering Department of the C.M. & St. P Ry. as well as from Mr. Fred G. Heuchling of the West Park Board. Turneaurer and Maurer's work on concrete was adopted as the standard reference. Reference was also made to Baker's "Masonry Construction", the Engineering Record, Railway Gazette and a number of drawings of arches erected by the City of Chicago. Wadell's "De Pontibus" was consulted in determining the thickness of the arch rib.

61115

LOCATION.

The location of the proposed three span arch is at the crossing of Humboldt Blvd. with the branch of the C.M. & St. P. Ry known as the Bloomingdale Road. The location of said crossing is about two blocks north of North Avenue. The territory under discussion is under the jurisdiction of the West Park Commissioners.

KIND OF CROSSING OR SUBWAY.

By ordinance passed by the council of the City of Chicago on June 27, 1910, the C.M. & St. P.Ry. is forced to cross over Humboldt Blvd. by means of a bridge on or before June 27, 1915. The clear head room must be 13'-6" from the street level. According to the Engineering Department of the West Park Commissioners the Railway will be allowed to depress the Boulevard, which is 250 feet wide and consists of two streets 50 feet, one main boulevard of 75 feet and two 60 foot parkways, two feet and still not interfere with the sewer grades. The ordinance of the city does not provide for any particular type of bridge. However, it allows the Railway to place piers in the center of center drive way.

PURPOSE OF THESIS.

It was the desire of the West Park Commissioners to have a bridge erected which would be in keeping with the purpose of the parkway; namely, pleasing to the eye. The only bridge which possessed the required beauty was a system of arches. Such a plan, however, the Railway claimed was too expensive to consider. The purpose of our thesis, then, was to investigate the practicability of crossing Humboldt Blvd. by means of a series of arches and to draw up a plan for the crossing.

DETERMINATION OF THE ARCHES.

As seen on plate 1, the parkway consists of three driveways and two parkways. After drawing up arches of various spans it was decided that the most pleasing effect, other points considered, could be obtained with a center arch of 90'-0" span and two end arches of 70'-0" span. In determining the spans of the arches it was necessary to provide for the requisite head room and still keep the elevation of the base of rail as low as possible. The intrados of the

one arch was a semi-ellipse whose major axis is 90'-0" and the semi-minor axis is 15'-0". The extrados of the same arch was a segment of a semi-ellipse whose major axis is 112'-0" and the semi-minor axis is 18'-0". The neutral axis of the rib, same amount of steel being used in the top and bottom, was a five centered arch as shown on Plate 5. The intrados of the other arches was a semi-ellipse of 70'-0" major axis and 13'-6" semi-minor axis. The extrados of these arches was a semi-ellipse of 88'-0" major axis and 16'-6" semi-minor axis. The neutral axis was a five centered curve as shown on Plate 2. After consulting the designs of a large number of arches it was decided to use 1 1/8" square bars at four inch centers for the large arch. These proved to be too heavy and the size finally used were 7/8" square bars at 6" centers. For the small arch 1 1/8" bars at four inch centers was tried but finally changed to 3/4" square bars at 6" centers.

METHOD OF DESIGN.

In the analysis of the arch ribs, the elastic theory, as developed by Turneaure and Maurer in "Principles of Reinforced Concrete Construction", was used.

The arch was analysed for two conditions; namely, (1) live load on half the span, and (2) live load on the entire span. Besides these conditions the rib was checked for dead load only.

The following general data was used:

Weight of earth fill:120 pounds per cubic foot.

Weight of concrete :150 pounds per cubic foot.

Weight of ballast:120 pounds per cubic foot.

The ratio of the modulus of elasticity of steel (E= 30,000,000) to that of concrete (E=2,500,000) was assumed to be 12.

The allowable stress of steel was taken as 15,000 pounds per square inch.

The allowable stress on concrete was taken as 750 pounds per square inch for a 1:2 1/2:5 concrete or better.

The bond strength was assumed as 100 pounds per square inch, since all bars used were deformed bars.

The unit shear on plain concrete was taken as 30 pounds per square inch.

For reinforced members the unit shear was taken as 75 pounds per square inch.

LIVE LOAD.

The method of determining live load was as follows: According to specifications (C.M. & St.P. Ry.) " For spans over twenty feet and where fill under rail is more than two feet a uniform load equivalent to E55 loading should be employed."

Weight of engine equals 177 1/2 tons. Base of engine equals 54.5 feet. Weight per foot of track

$$\frac{177.5 \times 2000}{54.5} = 6500 \text{ pounds.}$$

The tie is 8 feet wide. The load is assumed to spread 6" horizontal for each vertical foot of fill.

Therefore at crown where fill is 2 feet, the load per square foot equals $6500/8+1 = 724$ pounds.

The specifications require a 50% impact stress. Therefore the total live load per square foot equals $724 + 50\%\ 724 = 1086$ pounds. This can be called 1000 pounds per square foot because the fill under the rail soon increases and hence increases the live load spread and reduces its intensity.

DESIGN OF ARCH RIB.

The arch ribs were analysed according to the "Elastic Theory" as explained by Turneaure and Maurer. The size of the ribs are given on Plates 2 and 5. The analysis was a combination of analytical and graphical solution. The analytical method was employed where-ever possible and checked graphically. The determination of the dividions of the arch ribs and the value of the thrusts were determined only graphically.

In the development of the "Elastic Theory" for beams it is assumed that the moment of inertia of the beam is the same thruout. In the application of this theory to arches it is necessary to assume that the ratio of the length of a division to the moment of inertia of a section at the center of said division be constant. Hence, in analyzing the arches the first thing necessary is to divide the ribs into such lengths that the ratio of length "ds" of each division to the moment of inertia ,"I" will be a constant.

To do this the large arch was divided into 30 equal divisions and the small arch into 20 equal divisions. The moments of inertia of "transformed" section at the centers of divisions were calculated.

A base line equal to the half rectified neutral axis was laid off and divided into the 15 or 10 equal divisions as the case might be. At the centers of these divisions ordinates were erected equal to the values of I at the corresponding sections. A smooth curve was drawn thru these points. An inclined line was now determined which, when repeatedly drawn from base to curve and back to base would finally divide the base into 15 or 10 divisions as the case might be. Then we had the values of the divisions of the arches such that ds/I would be a constant. This construction is shown on Plates 3 and 5.

The loads on each section were now determined and their points of application, relative to the center line at the crown, and also relative to the center of their particular section centers, determined. With these quantities the bending moment at the center of each section due to external loads was calculated. The moments on the right and on the left were determined. Knowing these the values of H_o, M_o and V_o were determined by the following equations, given by Turneaure and Maurer:

$$H_o = \frac{n \Sigma my - \Sigma m \Sigma y}{2\left[(\Sigma y)^2 - n \Sigma y^2\right]}$$

$$V_o = \frac{\Sigma\left[(m_R - m_L)x\right]}{2 \Sigma x^2}$$

$$M_o = -\frac{\Sigma m + 2H_o \Sigma y}{2n}$$

where

$n =$ number of divisions in half the arch

$m_r =$ bending moment on right half of arch due to external loads

$m_l =$ bending moment on left half of arch due to external loads.

$\Sigma m = \Sigma(m_r + m_L)$

$x =$ distance of center of division to crown of arch

$y =$ distance of center of division to neutral axis at crown.

$\Sigma =$ sign of summation.

$H_o =$ thrust at crown

$V_o =$ shear at crown which is positive when upward on left of the sections.

$M_o =$ bending moment at crown assumed as positive when causing compression in the upper fibres.

The bending moment at any section may be calculated from the equation

$$M = M_o + m + H_o y \pm V_o x$$

where

M = total bending moment at section

M_o = bending moment at crown

$H_o y$ = bending moment at section caused by H_o.

$V_o x$ = bending moment at section caused by V_o.

$V_o x$ is negative or positive depending on the direction of rotation caused by V when compared with that caused by H_o.

Knowing the value of H_o and V_o the stress diagrams were drawn as shown on Plates 3,4,6 and 7. The value of V_o was laid above or below the center of the load line so that the line joining the pole with the end of V_o had the same inclination as the resultant of H_o and V_o plotted at the crown of the arch. From these the thrusts and shears at each section were found by drawing lines tangent and normal to the neutral axis at the section considered. By means of the stress diagram a moment polygon was drawn and the eccentricities of the thrusts scaled off. These eccentricities were checked by dividing the total moment at the section by their respective thrusts.

Knowing the eccentricities and thrusts, the

stresses were determined from the equation

$$S = \frac{P}{A} + \frac{P\,e\,c}{I}$$

where

s = unit stress in square inches

A = area of transformed section in square inches

P = thrust

e = eccentricity in inches

c = distance from neutral axis to outer fibre which equals $d/2$ in our case.

I = moment of inertia of transformed section.

TEMPERATURE.

According to the standards of the C.M. & St.P. Ry. a temperature provision for a rise of 15° F. and a fall of 35° F. must be made. The thrust at the crown is given by Turneaure and Maurer as

$$H_o = EI/ds \times \frac{ctln}{2\left[n\Sigma y^2 - (\Sigma y)^2\right]}$$

where

E = modulus of elasticity of concrete 2,500,000.

t = change in temperature in degrees Fahrenheit.

c = coefficient of expansion 0.000055 (C.M.& St.P. Ry.) and the other symbols are as given above.

From H we get the value of $M = -\frac{H_o \Sigma y}{n}$. From this we can determine the eccentricity of the thrust at the crown and determine the moment polygon.

The bending moment at any section equals $M = M_o + H_o\, y$.

The shear and thrust at all sections were determined by resolving H normal and parallel to the neutral axis at the various sections. This was done analytically. The stresses produced by change in temperature were tabulated together with the stresses due to live load and the maximum conditions determined for each section. The stress on the concrete was less than 600 pounds per square inch except at sections 1-4, 15 and spr. Here additional steel was added. See Plate 11. The unit shears were less than 30 pounds per square inch, except at sections 14 to springing. Here stirrups were placed as shown on Plate 11. Stirrups were placed the entire span of the arch to help space the rods.

Temperature joints were allowed in the retaining walls every 30 feet. A construction joint at the skewback provides for a temperature change in the mass piers and abutments.

DESIGN OF STIRRUPS.

Small Arch (See Plate 11.)

Design of Stirrups at Springing.

Maximum dead load and live load shear	47000
Temperature shear	54700
Total shear	101700

Area of concrete at springing = 6.6 x 144

Unit shear equals 101700/ 6.6 x 144 = 107 pounds per square inch.

107 - 30 = 77 pounds per square inch to be taken by stirrups.

Try 3/4" round hooked stirrups.

Let X" spacing (longitudinal at springing).

Use 6" spacing transversely.

Therefore 77 x 6 x X = 462X pounds to be taken by one stirrup.

Strength equals .44 x 15000 = 6610 pounds.

Therefore spacing = 6610/462 = 14.3 inches.

Now if we use double stirrups we can use a spacing of 2 feet.

Since the stirrups are practically normal to the arch ring we can figure them as vertical stirrups near the springing.

Stirrups at 10th Section.

Maximum deadload and live load shear	33,000
Temperature shear	27,300
Total shear	60,300

Unit shear = 60,300/5.1 x 144 = 82.75 pounds per square inch.

82.75 - 30 = 52.75 pounds per square inch to be taken by stirrups.

As before 52.75 x 6 x N = 313N

6618/313 = 21.5" spacing at section.

Therefore we will make spacing 1'-6".

Stirrups at 9th Section.

Maximum dead load and live load shear	8,000
Temperature shear	25,200
Total shear	[illegible]3,200

Unit shear 33,200/ 3.9 x 144 = 52.2 pounds per square inch.

52.2 - 30 = 29.2 pounds per square inch to be taken by the stirrups.

As before $29.2 \times 6 \times X = 175X$

$X = 6618/175 = 37.5''$

Therefore we will use 3'-0" spacing.

Stirrups at 8th Section.

Maximum dead load and live load shear	3,500
Temperature shear	19,500
Total shear	23,000

Unit shear = $\frac{23,000}{144 \times 3.5}$ = 45.6 pounds per square inch.

45.6 - 30 = 15.6 pounds per square inch to be taken by the stirrups.

As before $15.6 \times 6 \times X = 93.7X$

$X = 6618/93.7 = 70''$.

Therefore we will use 4'-0" spacing.

In our design we will use 4'-0" spacing from the 8th section to the crown.

DESIGN OF STIRRUPS.

Large Arch. (See Plate 11.)

Stirrups at Springing.

Maximum dead load and live load shear	44,800
Temperature shear	29,900
Total shear	74,700

$$\text{Unit shear} = \frac{74{,}700}{12 \times 5.95} = 104 \text{ pounds per square inch.}$$

104 - 30 = 74 pounds per square inch to be taken by the stirrups.

Use 3/4" round hooked rods.

Strength of one rod = .44 x 15000 = 6610.

Let X spacing longitudinally.

We will use 6" spacing transversely.

104 x 6 x X = 62.3X pounds to be taken by one stirrup.

$$\frac{6610}{623} = 10.6\text{" spacing.}$$

If we use double stirrups we can use a spacing of 1'-8".

Stirrups at 15th Section.

Maximum dead load and live load shear	41,200
Temperature shear	12,800
Total shear	54,000

$$\text{Unit shear} = \frac{54,000}{4.3 \times 12} = 104$$

Therefore we will use same spacing as before.

Stirrups at 14th Section.

Maximum dead load and live load shear	19,200
Temperature shear	9,000
Total shear	28,200

$$\text{Unit shear} = \frac{28,200}{3.9 \times 12} = 60 \text{ pounds per square inch.}$$

60 - 30 = 30 pounds per square inch to be taken by the stirrups.

As before 30 x 6 x X = 180X

X = 6610/ 180 = 37"

We will use 3 foot spacing.

Stirrups at 13th Section.

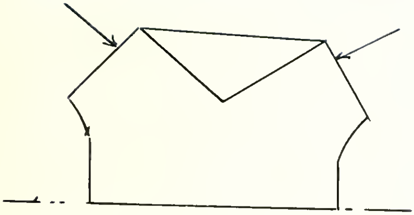
Maximum dead load and live load shear	[illegible],600
Temperature shear	[illegible],310
Total shear	12,410

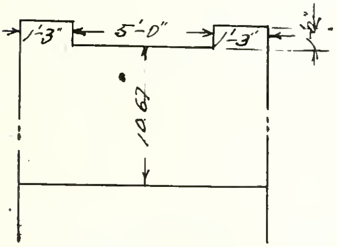
$$\text{Unit shear} = \frac{12,410}{3.7 \times 12} = 28 \text{ pounds per square inch.}$$

Therefore no stirrups are required, but we will use 4'-0" spacing from here to crown.

DESIGN OF PIERS.

From plate 9 it can be seen that the thrust due to live load on the half span of the large arch fell outside of the lines of the abutment. In order to take care of this thrust it was decided to partially fill in the opening between the skewbacks of the two arches. These pockets were formed as may be seen on plates 9 and 11. The size of this pocket was assumed and checked out. The pockets were made 5'-0" wide and the webs were made 2'-6" thick. This arrangement , considered as a foundation was checked at two separate places.





Center of gravity of section shown is 5.52 feet from AA. Moment of inertia of section about this gravity axis is 840 inches. Eccentricity of thrust 6.15, 1.15, 5.0.

$$S = \frac{Mc}{I} + \frac{P}{A} = \frac{Pec}{I} + \frac{P}{A}$$

$$= \frac{163000 \times 7.5 \times 6.15}{840} + \frac{163000 \times 7.5}{82.5}$$

$$S = 59800 \text{ pounds per square foot}$$

$$= 415 \text{ pounds per square inch.}$$

At another section about five feet from this above, eccentricity = 3.45 feet.

$$S = \frac{Pec}{I} + \frac{P}{A} = \frac{P}{A}(6e/d + 1)$$

$$= \frac{163000 \times 7.5}{8.1 \times 7.5 \times 144} (6 \times 3.45/8.1 + 1)$$

$$= 440 \text{ pounds per square inch.}$$

In order to drain the water from these pockets they will be built on a slight grade and drained by means of a 4" tile drain laid with open joints. This drain will connect to the sewers by means of an outlet thru the pier under ground.

Plate 10 shows the final dimensions of the pier. Surface area of pier = 406,13 square feet. Therefore, the weight of pier per lineal foot of same is 406,13 x 150 equals 61000 pounds. The center of gravity was found by cutting the pier out of stiff paper and balancing same on a point of a pair of dividers.

The weight of the pier was combined analytically with the weight of earth above the pier. Weight of earth equals 15000 pounds. Therefore the total equals 76,000 pounds.

Plate 9 shows the graphical combination of forces to determine the cases of maximum and minimum stress at the toe and heel of the pier.

Fiveconditions of loading were investigated.

Case (1). Live load on far side of large arch.

Thrusts = 163000 pounds and 86000 pounds.

Vertical pressure = 185000 pounds.

This falls on edge of middle third when effective base is reduced from 35.5 to 33 feet.

Unit pressure (185000/33) x 2 = 11200.

Spacing piles 2 feet centers one way and 2'-6" centers the other; the load on last pile 11200 x 5 = 56000 pounds or 28 tons. According to C. M. & St. P. Ry. we are allowed 25 tons on a pile. This overstress occurs only on the end piles. All other piles are understressed. Hence we will assume that the understressed piles will carry some of the load of the overstressed piles. The horizontal force = 90700 pounds. Coefficient of friction = 90700/185000 which is .49. Therefore this is safe against sliding,

Case (2) Live Load on near side of small arch.

Thrusts = 109000 pounds and 114300 pounds.

Eccentricity of resultant = 3.55 feet.

Vertical pressure 181000 pounds.

$$S = \frac{181000}{35.5}\left(1 + \frac{6 \times 3.55}{35.5}\right) = 8150 \text{ pounds}$$

$$S = \frac{181000}{35.5}\left(1 - \frac{6 \times 3.55}{35.5}\right) = 2000 \text{ pounds.}$$

Pressure of 25 tons per pile allowable stress equals $\frac{25 \times 2000}{5} = 10,000$ pounds.

Therefore this design is O.K.

Case (3). Live Load on far side of large arch and on entire small arch.

Vertical pressure = 220000 pounds.

Eccentricity = 0.

$$S = \frac{220000}{35.5} = 6,700 \text{ pounds.}$$ Therefore O.K.

Case (4). Live load on entire large arch.

Vertical pressure 226000 pounds.

This pressure falls on edge of middle third when effective base is reduced to 29.5 feet. Maximum pressure equals

$$\frac{226000}{29.5} \times 2 = 15300.$$

Allowable pressure = 10,000 pounds. Excess = [illegible] pounds per square foot.

$$\frac{5300 \times 5}{2000} = 13 \text{ tons on three end piles.}$$

We will make the same assumptions as in "Case 1".

Case (5). Live load on both spans.

Vertical pressure = 260000 pounds.

Eccentricity = 0.

$$\text{Pressure} = \frac{260000}{35.5} = 7500 \text{ pounds per sq. ft. O.K.}$$

Shear on Stepped Footings.

As seen on plate 9, the depth of last step is 3'-0". Assuming the pile to project 1'-0" into the concrete, (C. M. & St. P. Ry.) the depth available for shear is 24". Unit shear is 30 pounds per square inch.

Load on one pile is 25 tons. This is distributed over a length of 2'-0". Depth required to resist shear without reinforcement $= \dfrac{25 \times 2000}{30 \times 2 \times 12} = 91$ inches.

With reinforcing $= \dfrac{25 \times 2000}{75 \times 2 \times 12} = 28''$

Hence it will be necessary to reinforce our footings. Using stirrups inclined at 45 degrees to the vertical the area of stirrups required per foot = .8vbs/f cos

which equals = $\frac{.8 \times 75 \times 12 \times 18}{12000 \times .91}$ = 1.18 sq. in.

We will use 11/16" round stirrups, looped every 9".

Area per foot = 4 x $\frac{12}{18}$ x .37 = 1.25 sq. in. O.K.

Reinforcing for Bending Moment.

M = 50000 x 1.5 x 12 = 900,000 inch pounds.

M = 900,000/2 = 450,000 inch pounds.

Depth to steel 36", 12", 3", 21".

p = .007 (from C.M.& St. P. Ry.)

Arear of steel 12 x 21 x .007 = 1.76 sq. in. per foot.

Therefore we will use 13/16" square bars at 4 1/2" centers.

GRAPHICAL SOLUTION OF ABUTMENT.

The thrust at the abutment at end of 70'0" span arch due to live and dead load on entire span was 167,000 pounds, which was greater than that due to any other loading. This thrust was resolved graphically with the vertical pressures due to live load over the abutment, earth fill and weight of abutment. (See Plate 6).

Thrust of arch =167000

Earth fill =38200

Live load =24000

Weight of abutment =38500.

The final resultant was 250000 pounds, giving a vertical component of 208,000 pounds.

The horizontal component would cause the abutment to slide. The resisting force is

208000 x .4 = 82,000 pounds, horizontal pressure due to vertical force.

$$\frac{34 \times 120 \times .3}{2} \times 34 \times 3 = 62,500$$ pounds, horizontal pressure to to earth. This gives us a total of 144,500 pounds, which is greater than 143,000.

The foundation is sandy and as we have a unit pressure of 208000/28 = 7450 pounds, we will use piles.

Spacing the piles as shown in pile plan, Plate 10, we have a pressure per pile of $\frac{208000 \times 32}{11 \times 15} = 40,300$ pounds if the pressure were uniform.

The maximum pressure per pile will be

$$S = 5\left(\frac{208000}{28} + \frac{6 \times 208000 \times 2.5}{1 \times (28)^2}\right) = 56800 \text{ pounds.}$$

There will be no need of putting steel at this point as the abutment is 14'-0" thick over these piles.

On the heel the maximum pressure will be due to the live load on the right half. The resultant pressure was 180000 pounds, giving a vertical pressure of 147000 pounds with an eccentricity of 2.85 feet.

The unit pressure at the heel will be

$$S = 147000/28 + (6 \times 147000 \times 2.85)/((28)^2 \times 1).$$

$$= 8450.$$

As one pile takes pressure of five square feet the pressure on piles at heel will be 34250 pounds.

$$M = wl = 34250\ (6\ 1/2 + 4 + 1\ 1/2)$$

$$= 411000 \text{ ft. lbs.}$$

$M = f_s Ajd$

$12 \times 411000 = 15000 \times A \times .87 \times 5.75 \times 12$

Therefore $A = 5.47$ sq. in. for width of 2 1/2 feet/

$A = 5.47/2.5 = 1.09$ for one foot wide.

We will use one inch square bars at 5" cdnters.

$$\text{Length for bond} = \frac{1 \times 15000}{70 \times 4} = 4'\text{-}6".$$

For the final dimensions of abutment see plate 10.

DESIGN OF RETAINING WALLS.

In order not to throw eccentric stress on the arch ring, the retaining walls were not considered as cantilevers walls. They were designed as gravity walls for 10 feet on either side of the center line. From this point the same section was used and steel rods were run from one wall to the other to keep them from being pushed out.

We will figure this wall to withstand a fill of dry sand. Then from graphical solution of earth pressure, (page 62) at a depth of 10 feet, the following

weight of sand 120 pounds per cubic foot

30° equals the angle of repose.

$(p+q)/2 = 1470.$ $(p-q)/2 = 730.$

$p = 2205$ and $q = 735.$ $q/p = .327$

Therefore to obtain the horizontal thrust on the wall at any depth we will multiply the vertical pressure by .327

DESIGN OF WALL AT INTERSECTION OF ARCHES.

Here we have the maximum fill of 12 feet. We also have a live load of 1000 pounds per square foot. Therefore the pressure at depth of 12 feet

= 12 x 120 + 1000 = 2440 pounds per square foot.

2440 x .327 = 800 pounds per square foot horizontal thrust at depth of 12 feet.

$$\therefore \frac{800}{2} \times 12 = 4800 \text{ pounds total thrust.}$$

Area of 1 1/2" round rod = 1.767 sq. in.

Strength is 1.767 x 15000 = 26,500 pounds.

$$\frac{26500}{4800} = \text{about 6 feet spacing.}$$

Therefore we will make 3 center rods 1 1/2" at 6 feet spacing.

Next we will try a section at a distance of 11 feet from the center line of the arch.

Fill = 8'-6"/

Therefore 8.5 x 120 = 1020 = pressure due to fill.

1000 + 1020 = 2020 = pressure(total)

2020 x .327 = 656 pounds per square feet horizontal, pressure at depth of 8.5 feet.

Total thrust = (656/2) x 8.5 = 2790 pounds.

Strength of one 1 1/2" round rod 26,500

26500/2790 = 9.5 foot spacing at this point.

Next figure for transvers rods at a point where the wall has a six foot fill.

Pressure at a depth of 6 feet = 6 x 120 = 720 pounds.

Total vertical pressure = 1000 + 720 = 1720 pounds.

Unit thrust = .327 x 1720 = 562 pounds per square foot.

Total thrust = (562/2) x 6 = 1685 pounds.

Spacing of 1 1/2" round rods $\therefore = \frac{26500}{1685} =$ 15.7 feet.

DETERMINATION OF CONCRETE QUANTITIES.

Abutments.

Surface area 390 sq.ft. (Page)

There are two abutments 32'wide.

Amount of concrete $\frac{390 \times 32 \times 2}{27}$ 925 cu. yards.

Piers.

Surface area 259.13 sq. ft.

Amount of concrete $\frac{259.13 \times 12 \times 2}{27} = 615$ cu. yds.

Concrete in braces forming pockets between skewbacks

$$\frac{58.5 \times 12 \times 2}{27} = 52 \text{ cu. yards.}$$

58.5 sq. ft. = equals surface area of face

12 feet equals total thickness of braces in 32'-0".

Total concrete = 615 + 52 = 667 cu. yds.

Arch Ribs.

Surface area small arch = 22161/150 = 147.6 sq. ft.

22161 = weight of rib per lineal ft. (Page)

Surface area of large arch = 25684/150 = 171 sq.ft.

25684 = weight of rib per lineal foot. (Page)

Rib area 171 2 @ 147.6 = 466.2 square feet/

$$\text{Volume} = \frac{466.2 \times 32}{27} = 552.5 \text{ cubic yards.}$$

Spandrel Walls.

Surface area small arch = 27033/120 = 225 square feet.

27033 = weight of earth fill on arch (Page)

Surface area large arch = 27775/100 = 277.75 sq. ft.

27775 weight of earth fill (Page).

Total surface area for two walls = 2(277.75 2@ 225)

equals 1455.5 square feet.

Maximum thickness of wall = 2'-3".

Volume of wall if it were of such entire thickness

$$\frac{1455.5 \times 2.25}{27} = 121.3 \text{ cubic yards.}$$

Due to batter on wall we must subtract 11/12 x 3/2 x 500/27

or 15.3 cubic yards.

Therefore volume of walls = 121.3 - 15.3 = 106 cubic yards.

Railings.

For a solid railing volume equals 3.5 x (90 2 x70)2/27

or 59.5 cubic yards.

Subtract 2 x 1/2 x 1/2 x 155/27 x 2 = 5.75 cu. yards for

openings in railings.

Therefore volume of railing = 59.5 - 5.75 = 53.75 cu.yds.

The concrete in the piers and abutments need not be the best. This is mass construction and a 1: 3: 6 concrete will do. The concrete in the arch rib must be very good. This will be 1:2:4. The concrete in the spandrel walls and railing while not mass concrete need not be as good as that in the ribs. Therefore it will be 1:2 1/2 : 5.

Conclusion.

As stated in the beginning of this paper, the purpose of this thesis was to investigate the practicability of the construction of an arch bridge. The conclusion must be viewed from the side of the Railway and also from the side of the West Park Commissioners.

As far as the West Park Commissioners are concerned there is no doubt but that the aethetic qualities of such construction are far superior to that of plain spans.

From the point of view of the Railway, the first great objection is the fact that arch construction increases the elevation of the base of rail by 2 or 2 1/2 feet. This necessitates that every train crossing over Humboldt Boulevard would have to be raised two feet or more than if a plain horizontal span, plate girder bridge was constructed. This quantity could not easily be calculated in dollars and cents, but the fact still remains that the additional power consumed would be enormous. Again the form work required in connection with arches would far exceed that of a plain span. Again the size of piers and

abutments is excessive. This however, might be balanced by the large number of small piers required in plate girder construction.

It would be impossible to make a further comparison between the cost of an arch bridge, such as is proposed in this thesis and a plate girder or slab concrete bridge, without figuring their actual costs.

SMALL ARCH 70 Foot Span.

Properties of Divisions.

Sec NO.	Depth in Inches.	I_c	I_s	nI_s	I $I_c + nI_s$	I in Ft.	Ds in Ft.	Final Deapth
I	36.00	46656	439	5260	5I9I6	2.50	2.35	3.00
2	36.00	46656	439	5260	5I9I6	2.5	2.35	3.05
3	36.60	4902 8	458	5500	54528	2.63	2.40	3.I0
4	38.40	56623	5I8	62 00	62 823	3.03	2.45	3.I5
5	40.85	68000	602	7220	7522 0	3.63	2.65	3.25
6	43.2 5	8II00	693	83I0	894I0	4.32	2.95	3.30
7	46.20	986II	8I6	9800	I084II	5.2 7	3.45	3.35
8	5I.00	I3265I	I034	I2400	I4505I	7.0I	4.00	3.50
9	58.50	20020I	I435	I7200	2I7402	I0.50	5.20	3.90
I0	70.50	350402	2I92	26222	376623	I8.20	I0.I5	5.I0
Spring.	79.20	495000	2I84	34I00	529000	2 5.5		6.60

SMALL ARCH.

Table For Finding Points of Application of Pressures

Sec. No.	Weight of Concrete #	Lever Arm.	Concrete Moment.	Wt. of Earth. #	Lever Arm.	Earth Moment	L.L.over Section	Lever Arm.	Live Load Moment
I	I060	I.2'	I270	9I5	I.25	II43	2500	I.25'	3I25
2	I068	3.6	3850	908	3.68	3340	2400	3.70	8890
3	II05	6.05	6690	967	6.I5	5950	2500	6.I5	I5360
4	II60	8.5	9870	I040	8.60	8950	2550	8.70	22200
5	I283	II.I	I4200	II68	II.25	I3I30	2680	II.25	30200
6	I445	I3.85	20I00	I420	I4.I0	20000	3000	I4.I0	42400
7	I730	I7.02	29450	I880	I7.30	32600	3500	I7.35	60700
8	2I00	20.60	43250	2495	2I.I0	52600	4000	2I.I0	84500
9	3030	25.05	75900	3940	25.60	I0I800	5200	25.70	I33500
I0	8I80	32.75	268000	I2300	34.I0	420000	I0680	3I.65	383000
	22I6I			27033			39010		

SMALL ARCH. 70 Foot Span.

Distances From Center Of Span To Points Of Application Of Loads.

Sec.	Right. Live And Dead Load			Left. Dead Load Only.		
No.	Total Loads	Total Moments	Dist. From Center to ctr. gr. Loads	Total Loads	Total Moments	Dist. From Center to Ctr. Gr. Loads
I	4475#	5538	I.24'	I975#	24I3	I.22'
2	4376	I6080	3.68	I976	7I90	3.64
3	4572	28000	6.I3	2072	I2640	6.I0
4	4750	4I020	8,65	2200	I8820	8.56
5	5I3I	57530	II.2I	245I	27330	II.I5
6	5865	82500	I4.07	2865	40I00	I3.98
7	7II0	I22750	I7.28	36I0	62050	I7.20
8	8525	I80350	2I.00	4595	95850	20.85
9	I2I70	3II200	25.60	6970	I77700	25.50
10	3II60	I026000	33.00	20480	688000	33.60

SMALL ARCH. 70 Foot Span

NO. Sec.	X	Y	X^2	Y^2	D.L.Only m. left.	L.L. & D.L. m. right	$(M_1+M_r)Y$	$(M_r-M_1)X$
I	I.2	.00	I.44	.00	00000	00000	000	00000
2	3.6	.05	I2.96	,003	4700	I0580	764	2II50
3	6.05	.20	36.60	.04	I4230	3I900	9230	I07000
4	8.5	.38	72.05	.I44	28685	64400	35350	304000
5	II.08	.60	I2I.I5	.36	49746	II0650	96250	675000
6	I3.I3	I.00	I92.00	I.00	7942I	I75I50	25457I	I328000
7	I7.02	I.50	290.00	2.25	I2I688	265650	28I000	2450000
8	20.60	2.25	425.00	5.05	I82428	393450	I295000	4347000
9	25.06	3.50	628.00	I2.22	278I28	589550	3037000	780I000
I0	32.75	6.95	I072.00	48.30	495928	I02I550	I0546000	I72I4000
Σ		I6.43	285I.20	69.37	I254954	2662880	I5555I65	34247I50

Small Arch 70 Foot Span.

NO. Sec.	H_0Y	V_0X	Bending MOments Left.	Right	Thrusts Left.	Right.	Sec. Dist. Left.	Right.
I	0000	7200	+II920	+26320	I07500	I08000	+.II	+.22
2	5380	2I600	- I800	+35520	I07400	I04300	-.02	+.30
3	2I500	36000	-99I0	+45020	I07700	I07300	-.II	+.38
4	40900	5I000	-I9665	+46620	I08000	I07500	-.22	+.39
5	64600	66800	-32826	+39870	I08500	I08200	-.225	+.33
6	I07600	83800	-36505	+25370	I08600	I08800	-.350	+.26
7	I6I200	I02000	-43368	+I6670	I09200	I09800	-.400	+.I5
8	242000	I23500	-44808	- 8830	I09800	III700	-.4I5	-.II
9	377000	I53500	-35008	-40430	II0700	II4200	-.300	-.43
I0	748000	I96000	+75I92	-58439	I09500	II8000	+ .620	-.55
Spring	I076000	2I9000	+2I33I2	-30430	II4300	I33600	+ I.870	-.25

Small Arch 70 Ft, Span.

Total Shears for Live Load on Right Half of Span

Sec. No.	Total Shear Left	Total Shear Right
1	6000	6000
2	4000	6000
3	3000	3800
4	2600	2800
5	1800	100
6	800	3200
7	0	3000
8	3500	2000
9	8000	2100
10	28000	15000
Spring.	38600	20700

Small Arch 70 Foot Span.
Dead Load Only.

Sec. NO.	M_l & M_r	Y.	$(M_l+M_r)Y$	$(M_r-M_l)X = 0$
1	0000	.00	000	
2	4700	.05	470	
3	14230	.20	5700	
4	28685	.38	21800	
5	49746	.60	59600	
6	79421	1.00	158500	
7	121688	1.50	364500	
8	182428	2.25	821000	
9	278128	3.50	1950000	
10	495928	6.95	6900000	
	2509908	16.43	13256560	
Spring	662828	10.00	10281570	

$$H_0 \quad \frac{n\Sigma my - \Sigma m \Sigma y}{2((\Sigma y)^2 - n\Sigma y^2)} = \frac{-10 \times 10281570 + 2509908 \times 16.43}{} = 847.44$$

H_0 + 72,800#

$$M_0 = \frac{\Sigma m + 2H_0 y}{2n} = \frac{2,509,908 + 2 \times 16.43 \times 72800}{20}$$

$M_0 = +5995.40$ Say + 6,000#

$V_0 = 0$

Small arch 70 Foot Span.

Table For Live and Dead Load Over Entire Span.

NO. Sec.	M_r & M_l	$(M_r+M_l)Y$	$H_o Y$	Bending M. Right.	Thrusts. Right&Left.	Shear. R. & L.	Eccentricities Right & Left
I.	0000 0	00000	0000	+20000	I50000	0	.I34
2	I0580	I0580	7500	+I6920	I50000	0	.II0
3	3I900	I2760	30000	+I8I00	I50000	0	.I2
4	64400	48900	57000	+I2600	I50000	900	,084
5	II0650	I33000	90000	- 650	I50500	I200	,008
6	I75I50	350300	I50000	-5I50	I5I500	2900	.040
7	255650	798000	225000	-20650	I53000	I000	.I25
8	393450	I770000	338000	-35450	I54000	2600	.250
9	589550	4I25000	525000	-44550	I56000	5500	.3I0
I0	I02I550 2662880	I4200000 2I448540	I042000	+40450	I56800	33000	.200
Spring	I344550	2689I000	I500000	+I75450	I67000	47000	I.05

M_l is the same as M_r $(M_r-M_l)\,X=0$

Small Arch 79 Ft. Span

Live Load on Entire Span

$V_o = 0$

$$M_o = \frac{-\Sigma m + 2H_o \Sigma y}{2n} = \frac{-5325760 \quad 2 \times 149800 \times 16,43}{20} = 20038$$

$$H_o = \frac{n \Sigma my - \Sigma m \; y}{2 \left[(\Sigma y)^2 - n \Sigma y^2 \right]} = 149800$$

Live Load on Right Half of Span

$$V_o = \frac{\Sigma (m_r - m_l) x}{2 \Sigma x^2} = \frac{-34247160}{2 \times 2\ 851.20} = -6005$$

$$H_o = \frac{10\ (-155555165) - (-3917834 \times 16.43)}{2 \left[(16.43)^2 - 10(69.3665) \right]} = 107593$$

$M_o = 19116.401$

Temperature Stress in Small Arch.
Due to Fall Of 30° F.

H_0 = Thrust at Abutment due to temp. change

$H_o = EI/ds. \frac{ctln}{2(n\,\Sigma Y^2 - \Sigma Y^2)}$ Referring to half of Arch

t is rise of fall in temp.

c is Coef. of expansion = .0000055

l is span

E is the Modulous of Elasticity 2,500,000 (C.M.&S.P. Spec)

n = no. of Divisions.

$\Sigma Y^2 = 64.36$

$(\Sigma Y^2)^2 = 269.94$

$I/ds = 1.39$ (ave)

$$H_0 = (144 \times 2,500,000 \times 139) \times \frac{.0000055 \times 30 \times 70 \times 10}{2(10 \times 69.3665 - (16.43)^2}$$

$H_0 = +68,100$

$$M_0 = -\frac{H_0\,\Sigma Y}{n} = \frac{68,100 \times 16.43}{10} = -112,000$$

For a Temp rise of 15° F. the temp. stresses will be 1/2 Of those due to a fall of 30° F.

FOr a Temp rise of 35° F. the temp stresses will be 35/30 th's Of those due to a fall of 30° F., but the sign of the stress in each fiber will be changed.

According to C.M.&S.P.R.R. spec. all Concrete structures must be designed for a Temp. rise of 15° and a fall of 35° F.

Table For Temperature Stresses In Small Arch.
Due To a Fall of 30 F.

Sec. NO.	Y	H_0Y	$M = M_0 + H_0Y$	Φ Angle with Horizon.	Trust = $H_0 \cos \Phi$	Shear = $H_0 \sin \Phi$	Max Concrete Stress Upper Fiber	Lower
I	.00	0000	112000	0°-40'	68050	810	-320	+616
2	.05	3405	108600	2°-20'	68000	2770	-290	+586
3	.20	13620	98380	3°-40'	67950	4350	-205	+493
4	.38	25900	86100	5°-00'	67900	5950	-119	+403
5	.60	40900	71100	6°-30'	67600	7710	- 51	+325
6	1.00	68100	43900	8°-20'	67300	9850	+ 47	+231
7	1.50	102150	9850	10°-20'	66900	12200	+114	+148
8	2.25	153100	41100	14°-00'	66000	16400	+222	+ 28
9	3.50	238000	126000	18°-30'	64500	21600	+353	-135
10	6.35	474000	362000	32°-10'	57600	36200	+392	-240
Spring	10.00	681000	569000	42°-45'	49900	46100	+496	-404

Small Arch

Table of Concrete Stresses.

Sec. No.	Fibre stress due to fall of 35° F.		Fibre stress due to rise of 15° F.		Fibre stress due to Live Load on entire arch.	
	Upper	Lower	Upper	Lower	Upper	Lower
1	-370	+720	+160	-308	-410	-244
2	-340	+685	+145	-293	-394	-258
3	-240	+575	+103	-247	-382	-252
4	-139	+460	+ 59	-201	-352	-276
5	-60	+379	+ 26	-163	-303	-307
6	+ 56	+269	-24	-116	-288	-315
7	+133	+173	-57	- 74	-265	-335
8	+259	+ 33	-111	-14	-222	-373
9	+413	-158	-177	+ 68	-192	-352
10	+458	-280	-196	+120	-242	-170
Spring.	+580	-471	-248	+202	-315	-37

Small Arch

Table of Concrete Stresses (cont.)

Sec. No.	Fibre stress due to Live Load on Right Half				Max. f_c
	Fibre on Left		Fibre on Right		
	Upper	Lower	Upper	Lower	
I	-2 98	-200	-346	-126	-780
2	-237	-253	-382	-84	-734
3	-193	-263	-411	-42	-651
4	-166	-286	-393	-56	-532
5	-133	-307	-344	-94	-470
6	-137	-297	-284	-150	-432
7	- 142	-288	-252	-178	-409
8	-102	-311	-170	-2 52	- 387
9	-121	-2 57	-38	-350	- 510
10	-215	-83-	-49	-252	-532
Spring	-290	+ 50	-89	-186	-656

Large Arch 80 Ft. Span

Table For Finding Points of Application of Pressures

Sec. No.	Weight of Concrete	Lever Arm	Concrete Moment	Wt. of Earth	Lever Arm	Earth Moment	L.L. over Section	Lever Arm	L.L. Moment
I	I040	I.2I	I258	675	I.I	743	2250	I.2	2700
2	I043	3.45	3600	684	3.5	2390	2260	3.5	7910
3	I055	5.8	6I30	70I	5.9	4I40	2280	5.82	I3280
4	I070	8.I2	8690	725	8.2	5940	2300	8.20	I6850
5	I078	I0.42	II220	773	I0.5	8I20	2350	I0.5	24700
6	II2I	I2.73	I4320	820	I2.9	I0580	2400	I2.9	3I000
7	II4I	I5.I5	I7300	920	I5.35	I4I00	2500	I5.32	38300
8	II95	I7.64	2II00	I000	I7.9	I7900	2520	I7.9	45200
9	I258	20.3	25500	II00	20.42	22450	2550	20.4	52000
I0	I388	22.8I	3I600	I352	23.I6	3I300	2850	23.I0	65800
II	I535	25.68	39400	I585	26.05	4I300	3000	26.05	78200
I2	I800	28.9	52000	2020	29.3	59200	3400	29.25	99700
I3	2II0	32.24	68400	2600	33.0	85800	3800	32.9	I25000
I4	2740	36.5	I00000	3760	37.2	I39800	4600	37.I	I70500
I5	6II0	42.4	259000	9060	44.0	398500	8200	43.5	356500

Large Arch 80 Ft. Span

Table For Finding Points of Application of Pressures

Sec. No.	Weight of Concrete	Lever Arm	Concrete Moment	Wt. of Earth	Lever Arm	Earth Moment	L.L. over Section	Lever Arm	L.L. Moment
I	I040	I.2I	I258	675	I.I	743	2250	I.2	2700
2	I043	3.45	3600	684	3.5	2330	2260	3.5	79I0
3	I055	5.8	6I30	70I	5.9	4I40	2280	5.82	I3280
4	I070	8.I2	8690	725	8.2	5940	2300	8.20	I6850
5	I078	I0.42	II220	773	I0.5	8I20	2350	I0.5	24700
6	II2I	I2.73	I4320	820	I2.9	I0580	2400	I2.9	3I000
7	II4I	I5.I5	I7300	920	I5.35	I4I00	2500	I5.32	38300
8	II95	I7.64	2II00	I000	I7.9	I7900	2520	I7.9	45200
9	I258	20.3	25500	II00	20.42	22450	2550	20.4	52000
I0	I388	22.8I	3I600	I352	23.I5	3I300	2850	23.I0	65800
II	I535	25.68	39400	I585	26.05	4I300	3000	26.05	78200
I2	I800	28.9	52000	2020	29.3	59200	3400	29.25	99700
I3	2II0	32.24	68400	2600	33.0	85800	3800	32.9	I25000
I4	2740	36.5	I00000	3760	37.2	I39800	4600	37.I	I70500
I5	6II0	42.4	259000	9060	44.0	398500	8200	43.5	356500

Large Arch 80 Ft. Span

Distances From Center of Span To Points of Application of Loads

Sec. No.	Live Load and Dead on Right Total Loads	Total Moments	Dist. of Cen. to ctr. gr. Load	Dead Load Only on Left Total Loads	Total Moments	Dist. of Cent. to cen. of gr. Loads
I.	3965	470I	1.188	1715	200I	1.165
2	3987	8509	2.13	1727	5990	3.47
3	4073	23550	5.84	1756	10270	5.85
4	4102	33480	8.18	1795	14630	8.15
5	420I	44040	10.5	185I	19340	10.4
6	434I	55900	13.85	194I	24900	12.8
7	456I	69700	15.3	206I	3I400	15.2
8	47I5	84300	I7.85	2195	39000	I7.8
9	4908	99950	20.4	2358	47950	20.3
I0	5590	I28700	23.0	2740	62900	22.95
I1	6I20	I58900	25.95	3I20	80700	25.9
I2	7220	2I0900	29.2	3820	III200	29.2
I3	85I0	279200	32.8	47I0	I54900	32.8
I4	IIIOO	4I0300	36.9	6500	239800	36.8
I5	23570	10I4000	43.3	I5I70	657500	43.4

Large Arch 80 Ft. Span

Sec. No.	X	Y	X^2	Y^2	D.L. Only M_1	I.L.&D.L. M_r	$(M_1 + M_r)Y$	$(M_r - M_1)X$
I	I.I	0	I.21	0	-000000	-0000000	000000	0000000
2	3.45	.05	II.9	.0025	- 9I60	- 39I0	654	I8I00
3	5.78	.10	33.4	.0I	- 32940	- II9I0	4485	I2I500
4	8.05	.I8	64.8	.0325	- 59940	- 23590	I5050	292000
5	10.43	.30	I09.0	.09	- 97720	- 40070	4I300	60I000
6	I2.75	.44	I62.5	.I94	-I44470	- 60620	9[illegible]200	I070000
7	I5.I5	.63	230.00	.396	- 203220	-86380	I82000	I770000
8	17.7	.93	3I4.0	.862	- 26I500	-II9040	354000	2525000
9	20.2	I.33	408.0	I.77	- 36II00	-I56400	689000	4I80000
I0	22.8	I.80	520.0	3.24	-46I020	-20I400	II90000	5920000
II	25.73	2.35	660.0	5.52	- 590030	-260000	I998000	8500000
I2	28.86	3.08	832.0	9.5	-746830	-332240	3520000	II950000
I3	32.4	4.07	I050.0	I6.6	- 948830	-426890	5600000	I6900000
I4	36.45	5.4	I330.0	29.2	-I2I3430	-553540	9540000	24050000
I5	42.30	8.4	1790.0	70.4	-166073$\underset{6}{0}$	-774640	20420000	37500000
		29.14	7516.8	137.317		-9841550		
Spring.45.35		10.65			-1990530	-947840	43644680	115397600

Large Arch 80 Ft. Span

Sec. No.	$H_o Y$	$V_o X$	Bending Moments		Thrusts		Ecc. Dist.	
			Left	Right	Left	Right	Left	Right
1	0000000	8420	+26280	+33120	151000	151000	+.174	+.219
2	7550	26450	+11890	+59540	151400	151200	+.078	+.394
3	15100	44300	- 6410	+61160	151400	151000	-.048	+.405
4	27200	61300	-23490	+63760	151400	151300	-.155	+.421
5	45300	79800	-39870	+62080	151600	151200	-.263	+.41
6	66500	97600	-57020	+54340	152000	151800	-.375	+.358
7	95300	116300	-72680	+43080	152600	152200	-.475	+.283
8	140500	135800	-79640	+49500	152 600	152800	-.522	+.324
9	201200	155000	-75700	+29600	153000	153400	-.495	+.193
10	272000	175000	-69700	+20680	153000	154400	-.455	+.134
11	355000	197000	-67300	-3330	153 600	155600	-.438	-.0214
12	465000	221000	-53540	-26130	154400	157000	-.346	-.166
13	651000	249000	-26190	-50130	154800	159200	-.169	-.315
14	815000	279500	+26660	-84230	155500	161000	+.171	-.522
15	1270000	325000	+205060	-31030	152600	165400	+1.34	-.1875
Spring.	1610000	350000	+346860	+ 4170	156800	176400	+2.52	+.0236

Large Arch 80 Ft. Span

Shears for Live Load on Right Half of Span

Sec. No.	Total Shear Left	Total Shear Right	Unit Shear Left	Unit Shear Right
1	9800	9800	22.7	22.7
2	4600	4200	10.6	9.3
3	6000	5200	13.7	11.9
4	7000	4000	15.6	9.1
5	6200	1600	13.9	
6	5700	1200	12.7	
7	4000	4000	8.95	
8	2000	3200	5.9	
9	400	4000	.84	
10	3600	4400	7.6	
11	3600	5000	7.4	
12	1000	5600	1.98	
13	5400	5600	10.7	
14	19200	1200	34.9	
15	41200	20800	66.6	33.6
Spring.	44800	21000	52.2	24.5

Large Arch 80Ft. Span

Dead Load Only

Sec. No.	$M_l = M_r$	Y	$(M_l + M_r)\,Y$
1	0000	0.0	000
2	3910	.05	391
3	11910	.10	2380
4	23590	.18	8470
5	40070	.30	24000
6	60620	.44	53300
7	86380	.63	109000
8	119040	.93	221000
9	156400	1.33	416000
10	201400	1.80	720000
11	260000	2.35	1223000
12	332240	3.08	2045000
13	426890	4.07	3470000
14	553540	5.40	5980000
15	774640	8.40	13000000
	3050630		27272541
Spring.	947840	10.65	20200000

$V_o = 0$

$$H_o = \frac{n\,\Sigma my - \Sigma m\,\Sigma y}{2\left([\Sigma y]^2 - n\,\Sigma Y^2\right)} = \frac{15(-2\ 7272541) + 2 \times 3050630 \times 29.14}{-2440}$$

$H_o = +95000$

$$M_o = -\frac{\Sigma m + 2H_o\,\Sigma y}{2n} = +19000$$

Large Arch 80 Ft. Span

Moments etc. for Live and Dead Load on Entire Span

Sec. No.	$M_l = M_r$	$(M_l + M_r)\ Y$	$H_o\ Y$	Moment Right=Left	Thrusts R = L	Shear R = L	Ecc. Dist. R = L	Unit Shear
1	0000000	0000000	0000000	+42700	205000	000	+.208	0
2	9160	916	10250	+43790	205000	50	+.214	.11
3	32940	6588	20500	+30269	205000	100	+.147	.23
4	59940	21550	36900	+19600	205000	200	+.0955	.45
5	97720	58622	61500	+ 6480	205000	2000	+.0316	4.46
6	144470	127000	90200	-11570	206000	5000	-.0561	11.05
7	203 220	256000	129000	-31520	206400	7000	-.153	15.45
8	261500	486000	190500	-28300	207000	4000	-.137	8.68
9	361100	960000	273000	-45400	208400	1600	-.218	3.37
10	461020	1660000	369000	-49320	208800	2600	-.236	5.47
11	590030	2770000	481500	-65830	209800	1800	-.314	3.69
12	746830	4600000	631000	-73130	211600	000	-.346	00
13	948830	7710000	834000	-72130	212600	2000	-.34	3.78
14	1213430	13110000	1108000	-62730	214800	14000	-.292	24.90
15	1660730	27900000	1720000	+101970	214800	42600	+.476	69.0
	6790920	59666676						
Spr.	1990530	42450000	21 90000	+242170	223200	48000	+1.08	55.0

Large Arch 80 Ft. Span

Live Load on Entire Span

$V_o = 0$

$$H_o = \frac{n\Sigma my - \Sigma m\Sigma y}{2\left[(\Sigma y)^2 - n\Sigma y^2\right]} = \frac{-15 \times 59666676 + 13381840 \times 29.14}{-2440} = 205000$$

$$M_o = -\frac{\Sigma m + 2 H_o \Sigma y}{2n} = -\frac{-13581840 \quad 2 \times 205000 \times 29.14}{30} = + 42700$$

Live Load on Right Half of Span

$$V_o = \frac{\Sigma (m_{r_2} - m_l)\, x}{2\Sigma x^2} = \frac{-115397600}{2 \times 7516.81} = -7675$$

$$M_o = \frac{-9841550 \quad 2 \times 151000 \times 29.14}{30} = +34700$$

$$H_o = \frac{15(-43644689) - (-9841550 \times 29.14)}{2\left[(29{,}14)^2 - 15 \times 137.82\right]} = 151000$$

Temperature Stress in Large Arch
Due to a Rise of 15^{o} F

H_o = Thrust at Abutment due to temperature change

$H_o = EI/ds.\ \frac{ctln}{2(n\ y^2 - y^2)}$ Reffering to half arch

t is rise or fall in temper.

c is Coef. of expansion =.0000055

l is span

E is the Modulus of Elasticity = 2,500,0000 (C.M.&St.P.Ry.)

n = number of divisions

$\Sigma y^2 = 137.82$

$(\Sigma y)^2 = (29.14)^2 = 848.7$

$I/ds = 1.46$ (ave.)

$$H_o = \frac{2,500,000 \times 1.46 \times 144 \times .0000055 \times 15 \times 90.7 \times 15}{2\ (15 \times 137.82 - 848.7}$$

$H_o = +24,200$

$$M_o = \frac{-H_0 \Sigma y}{n} = \frac{24,200 \times 29.14}{15} = -46,300$$

For a Tem. fall of 35^{o}F the tem. stresses will be 35/15 of those due to a rise of 15^{o}F., but the sign of the stress in each fibre will be changed.

According to C.M.&St.P.Ry.spec. all Concrete structures must be designed for a Tem. rise of 15^{o} and a fall of 35^{o}.

Table For Temperature Stresses in Large Arch 80 Ft. Span
Due to a Rise of 15° F.

Sec. No.	Y	H_o Y	$M = M_o + H_o Y$	θ Angle with Horizontal	Thust	Shear $H_o \sin θ$	Max. Concrete Stress Fibre Upper	Lower
1	0.00	000000	-46800	0° 4'	24200	281	+131	-231
2	.05	1210	-45590	1° 45'	24150	738	+127	-227
3	.10	2420	-44380	2° 10'	24125	915	+125	-225
4	.18	4360	-42 400	3° 00'	24100	1265	+117	-215
5	.30	7260	-39540	3° 3 0'	24075	1480	+100	-198
6	.44	10650	-36150	4° 6'	24050	1740	+ 84	-180
7	.63	15250	-31550	5° 30'	24025	2320	+ 61	-157
8	.93	2 2500	-24300	7° 0'	24000	2940	+33	-127
9	1.33	32200	-14600	8° 20'	23900	3500	0	-92
10	1.80	43600	-3200	10° 40'	23800	4480	-35	-55
11	2.35	57000	+10200	11° 40'	23650	4900	-75	-13
12	3.01	72700	+25900	13° 0'	23550	5450	-116	+30
13	4.07	98500	+ 51700	16° 20'	23200	6810	-179	+ 99
14	5.4	130800	+ 84000	21° 50'	22400	9000	-240	+166
15	8.4	203000	+156200	32° 0'	20400	12800	-341	+279
Spr.	10.65	258000	+211200	38° 10'	19000	14960	-248	+286

59

Large Arch 80 Ft, Span.

Table of Concrete Stresses.

Sec, No.	Fibre stress due to Rise of 15° F.		Fibre stress due to Fall of 35° F.		Fibre stress due to Live Load on Entire Arch	
	Upper	Lower	Lower	Upper	Upper	Lower
1	+131	- 2 3 1	+540	-306	- 592	-262
2	+127	-227	+530	-296	-595	-255
3	+125	-225	+525	-292	-541	-303
4	+117	-215	+502	-273	-494	-340
5	+100	-198	+462	-233	-438	-390
6	+ 84	-181	+422	-196	-373	-457
7	+ 61	-157	+368	-142	-304	-522
8	+ 33	-127	+2 96	-77	-314	-498
9	0	-92	+214	0	-255	-543
10	-35	-55	+128	+ 82	-243	-555
11	-75	-13	+ 30	+175	-189	-595
12	-116	+ 30	-70	+271	-268	-618
13	-179	+ 99	-231	+418	-172	-560
14	-240	+166	-387	+560	-199	-505
15	-341	+279	-650	+ 794	-528	-124
Spr.	-248	+206	- 482	+578	-506	+ 12

Large Arch 80 Ft. Span

Table of Concrete Stresses (cont.)

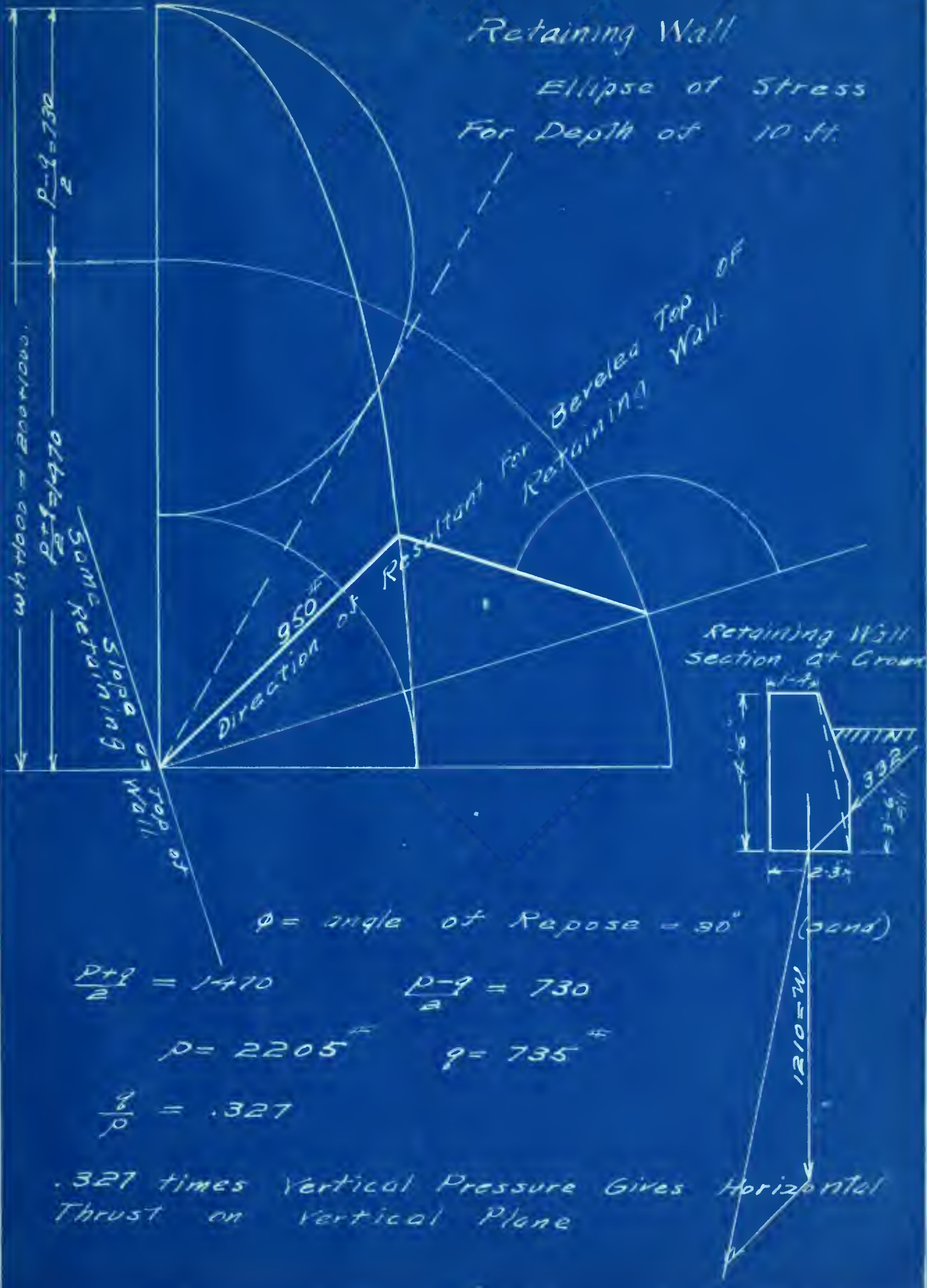
SEC. No.	Fibre Stress due to Live Load on Right Half Fibre on Left Upper	 Lower	 Fibre on Right Upper	 Lower	Max. f_c
1	-416	-214	-443	-187	-898
2	-261	-269	-543	-81	-892
3	-2 86	-336	-550	-68	-833
4	-216	- 400	-555	- 57	-767
5	-156	-456	-539	-73	-671
6	- 98	-514	-504	-108	-695
7	-53	-553	-451	-155	-710
8	-41	-559	-461	-139	-626
9	-53	-531	-3 87	-199	-635
10	-72	-514	-316	-229	-619
11	-79	-495	- 279	-299	-670
12	-130	-430	-212	-358	-688
13	-195	-337	-140	-410	-731
14	-319	-191	-61	-467	-692
15	-636	+178	-186	-310	-977
Spr.	+545	+199	-199	-191	-795

Library

Retaining Wall

Ellipse of Stress

For Depth of 10 ft.


(p-q)/2 = 730
wh+1000 = 2000+1000.
(p+q)/2 = 1470
Slope of Some Retaining Wall
Top of Wall
950#
Direction of Resultant for Beveled Top of Retaining Wall.
Retaining Wall Section at Crown
1-4
4-6
33 2/3
3-6
2-3
1210=W

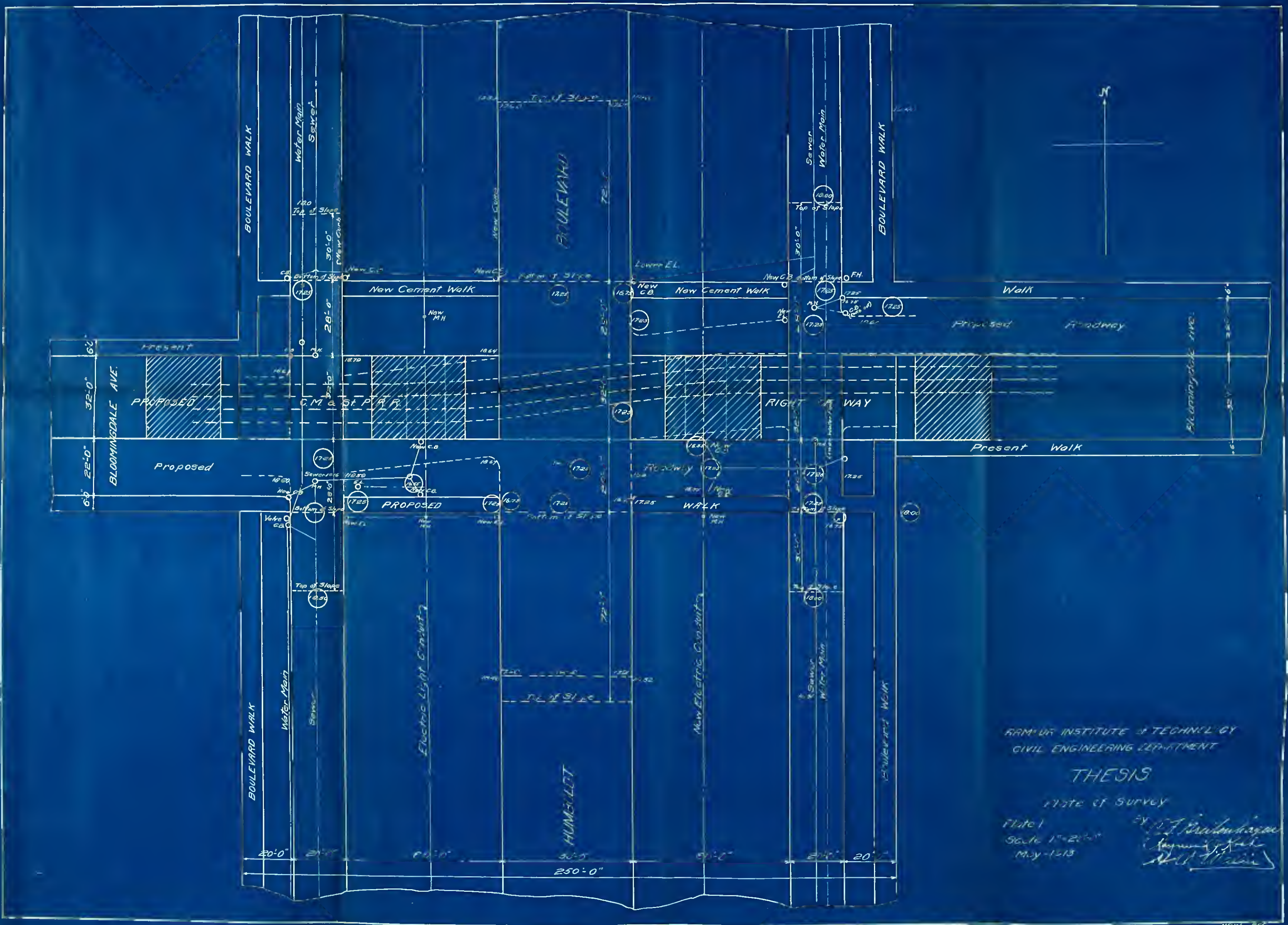
φ = angle of Repose = 30° (sand)

$\frac{p+q}{2} = 1470$ $\frac{p-q}{2} = 730$

$p = 2205\#$ $q = 735\#$

$\frac{q}{p} = .327$

.327 times Vertical Pressure Gives Horizontal Thrust on Vertical Plane [illegible] Library



ARMOUR INSTITUTE of TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

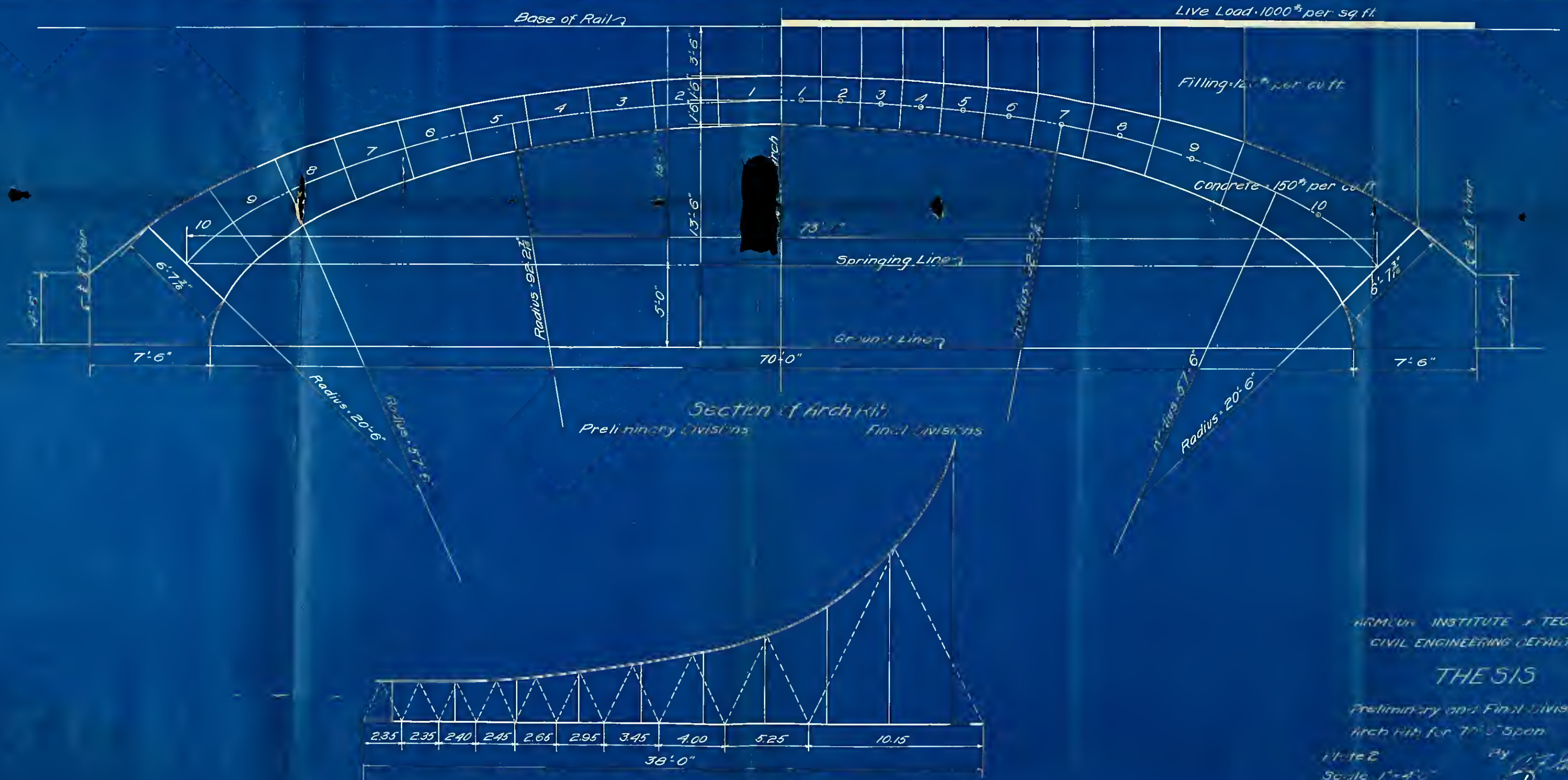
THESIS

Plate of Survey

Plate I
Scale 1" = 20'-0"
May - 1913

By C. J. [illegible]

HSW RFG



Section of Arch Rib

Preliminary Divisions

Final Divisions

Constant ds/I Diagram

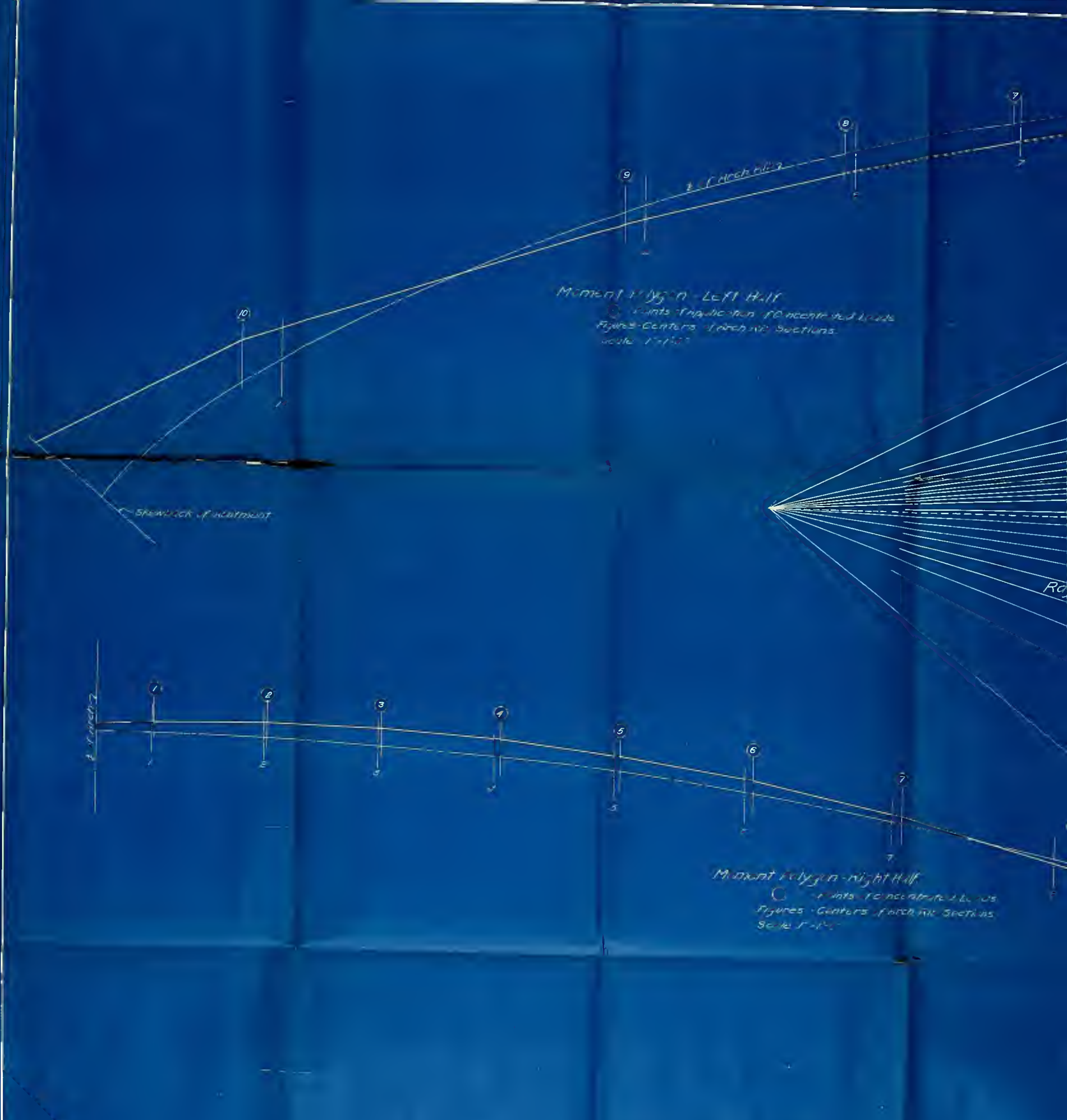
ARMOUR INSTITUTE of TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

THESIS

Preliminary and Final Divisions of
Arch Rib for 70'-0" Span

Plate 2
Scale 1" = 4'-0"
May - 1913

By



Moment Polygon - Left Half
○ Points of Application of Concentrated Loads
Figures - Centers of Arch Rib Sections
Scale 1"=10'

Moment Polygon - Right Half
○ Points of Concentrated Loads
Figures - Centers of Arch Rib Sections
Scale 1"=10'

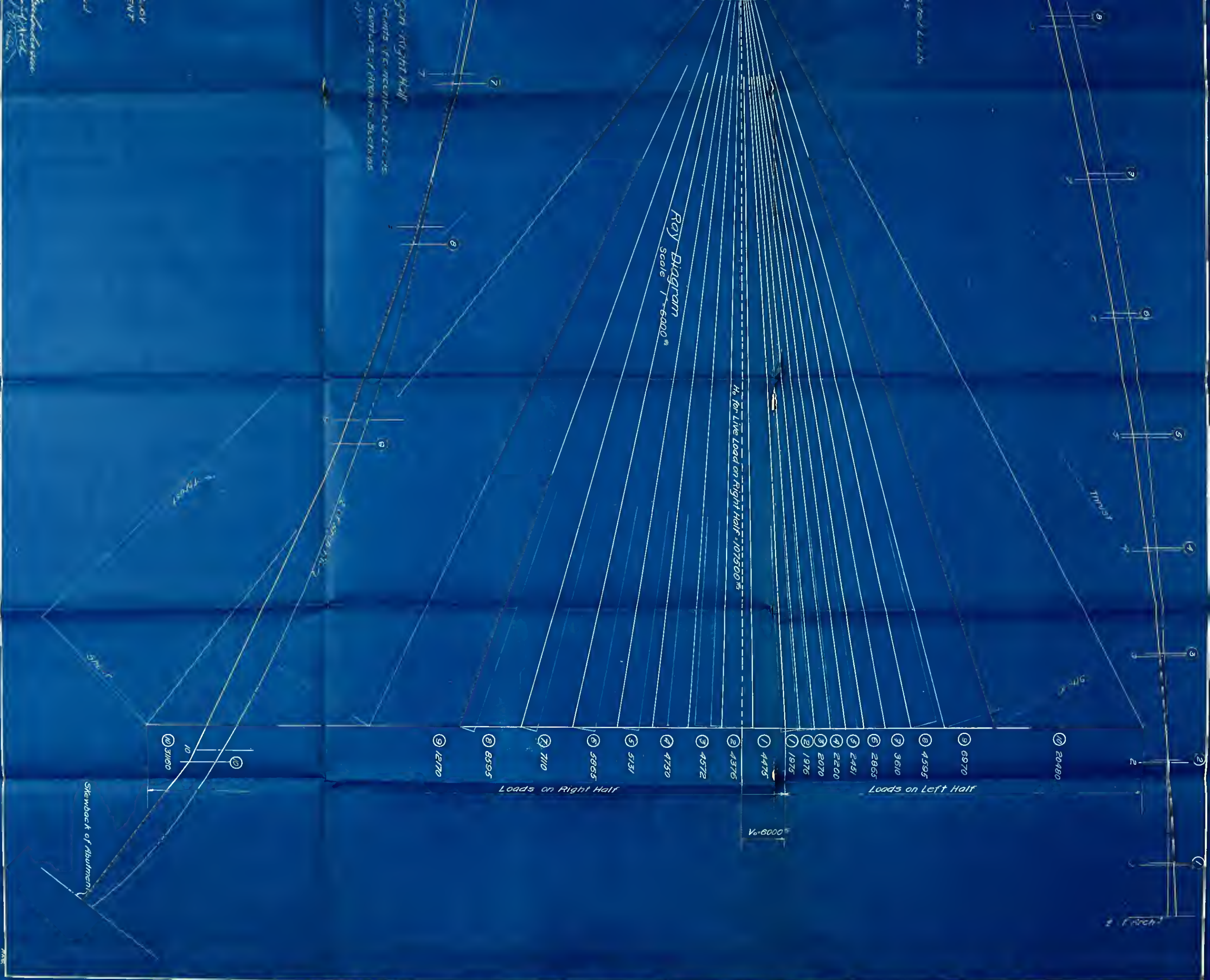
ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

THESIS

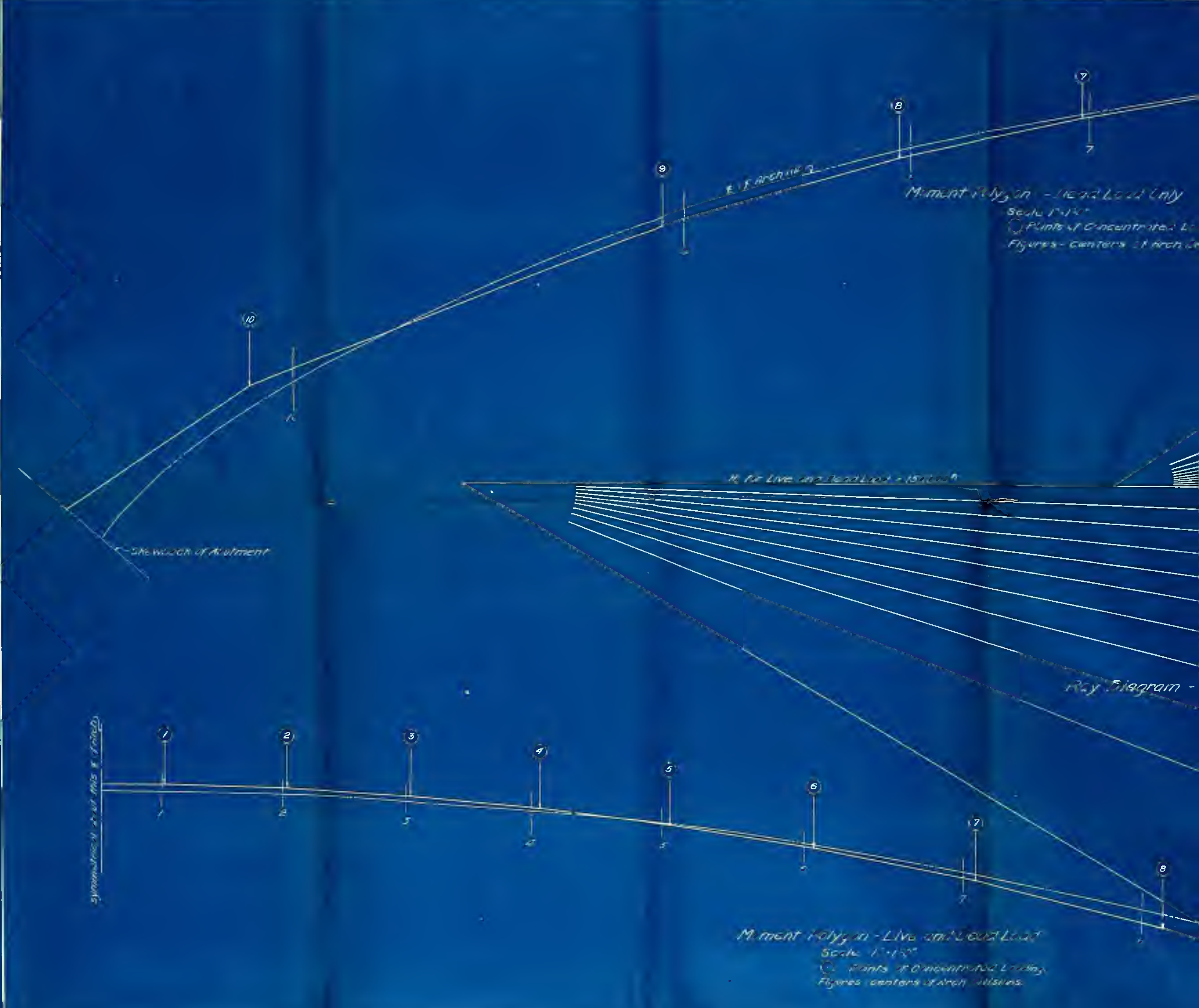
Stress Diagram For Live Load
on Right Half
100' Span

Plate 8
Scale Indicated
May — 1913

By



Ray Diagram
Scale 1" = 6000 #
H₀ for Live Load on Right Half = 107500 #
Loads on Right Half
Loads on Left Half
V₀ = 6000 #
⑩ 31160
⑨ 12170
⑧ 8595
⑦ 7110
⑥ 5865
⑤ 5131
④ 4750
③ 4572
② 4376
① 4475
① 1975
② 1976
③ 2070
④ 2200
⑤ 2451
⑥ 2865
⑦ 3610
⑧ 4595
⑨ 6970
⑩ 20480
Skewback of Abutment
Thrust
℄ of Arch
① ② ③ ④ ⑤ ⑥ ⑦ ⑧ ⑨ ⑩


Moment Polygon - Dead Load Only
Scale 1"=1'0"
○ Points of Concentrated L
Figures - Centers of Arch Se
E. F. Arch Rib
10
9
8
7
Skewback of Abutment
H for Live and Dead Load = 15[illegible]#
Ray Diagram -
Symmetrical about this E (Crown)
1
2
3
4
5
6
7
8
Moment Polygon - Live and Dead Load
Scale 1"=1'0"
○ Points of Concentrated Loading
Figures - Centers of Arch Divisions.

ARMOUR INSTITUTE of TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

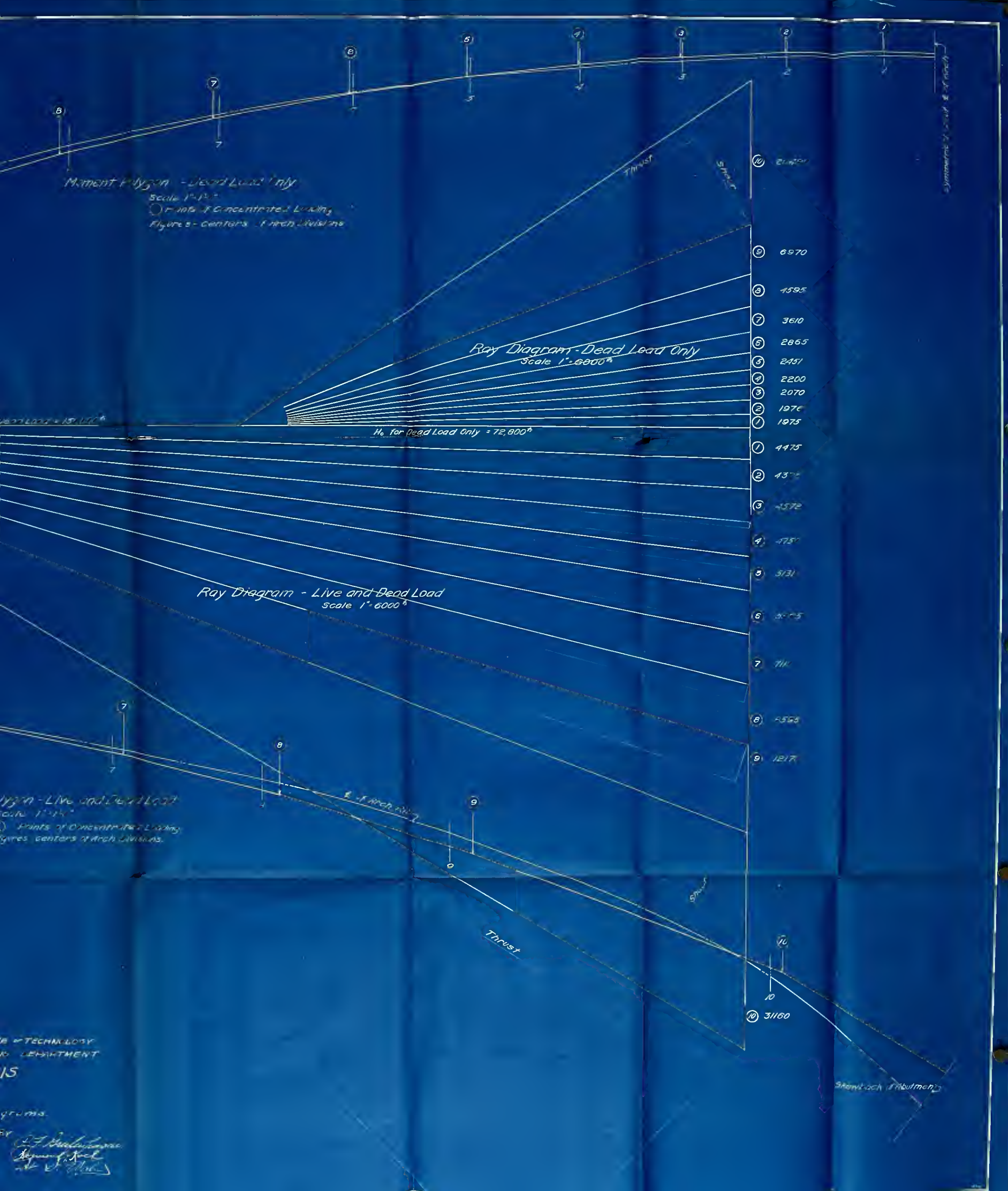
THESIS

75'-0" Span

Stress Diagrams.

By

Plate 4
Scale Indicated
May - 1913.


Moment Polygon - Dead Load Only
Scale 1"=1'-"
◯ Points of Concentrated Loading
Figures - Centers of Arch Divisions
Symmetrical about ℄ of Arch
1
2
3
4
5
6
7
8
Thrust
Shore
Ray Diagram - Dead Load Only
Scale 1"=6000#
⑩
⑨ 6970
⑧ 4595
⑦ 3610
⑥ 2865
⑤ 2451
④ 2200
③ 2070
② 1976
① 1975
H₀ for Dead Load Only = 72,800#
① 4475
② 437
③ 4372
④ 475
⑤ 5131
⑥
7 711
⑧
⑨ 1217
⑩ 31160
Ray Diagram - Live and Dead Load
Scale 1"=6000#
Polygon - Live and Dead Load
Scale 1"=1'-"
◯ Points of Concentrated Loading
Figures centers of Arch Divisions.
℄ of Arch Ring
9
10
Thrust
Shore
Skewback of Abutment
E OF TECHNOLOGY
DEPARTMENT
15
grams
BY

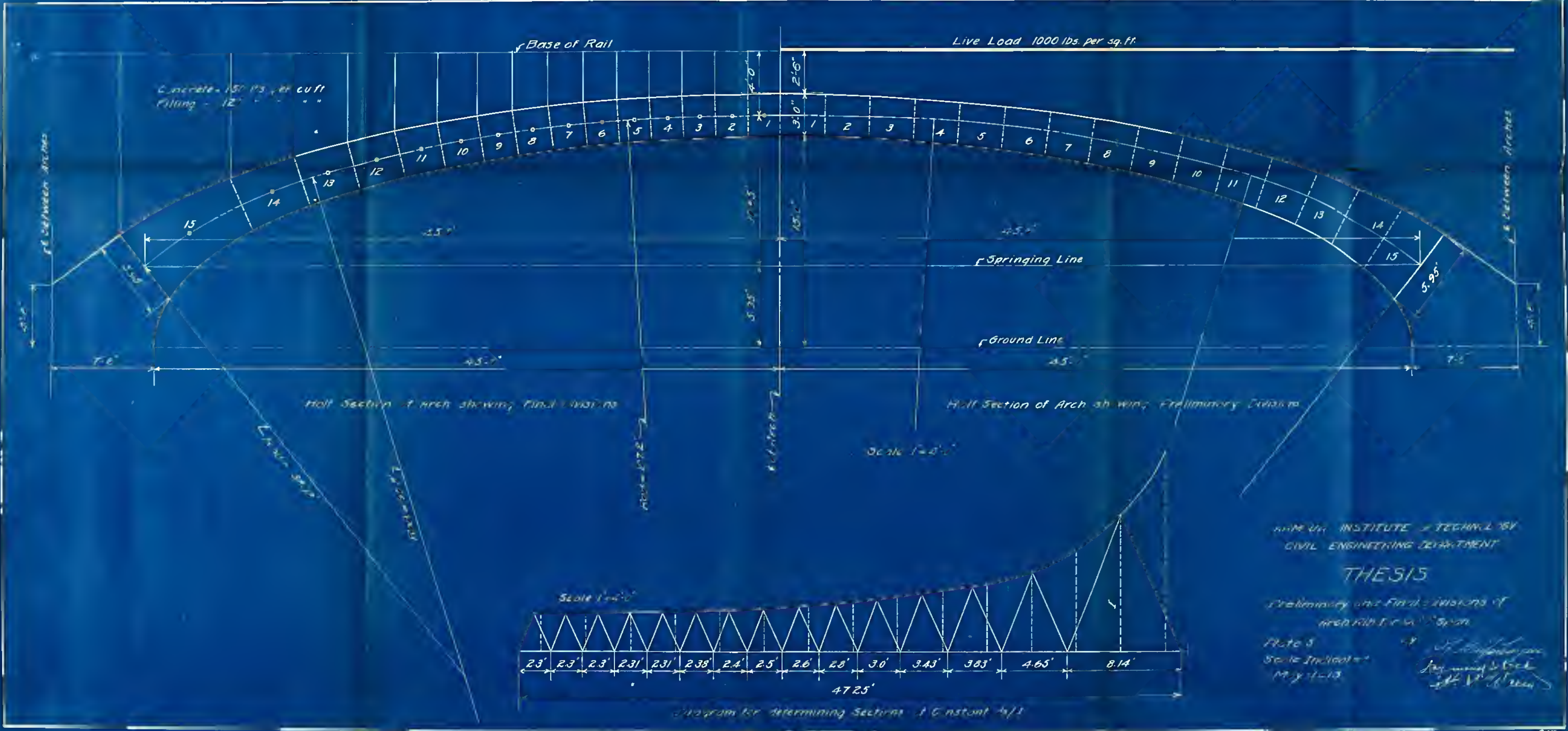


Diagram for determining Sections I Constant ds/I

ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

THESIS

Preliminary and Final Divisions of
Arch Rib for 90' Span

Plate 5
Scale Indicated
May 1-13

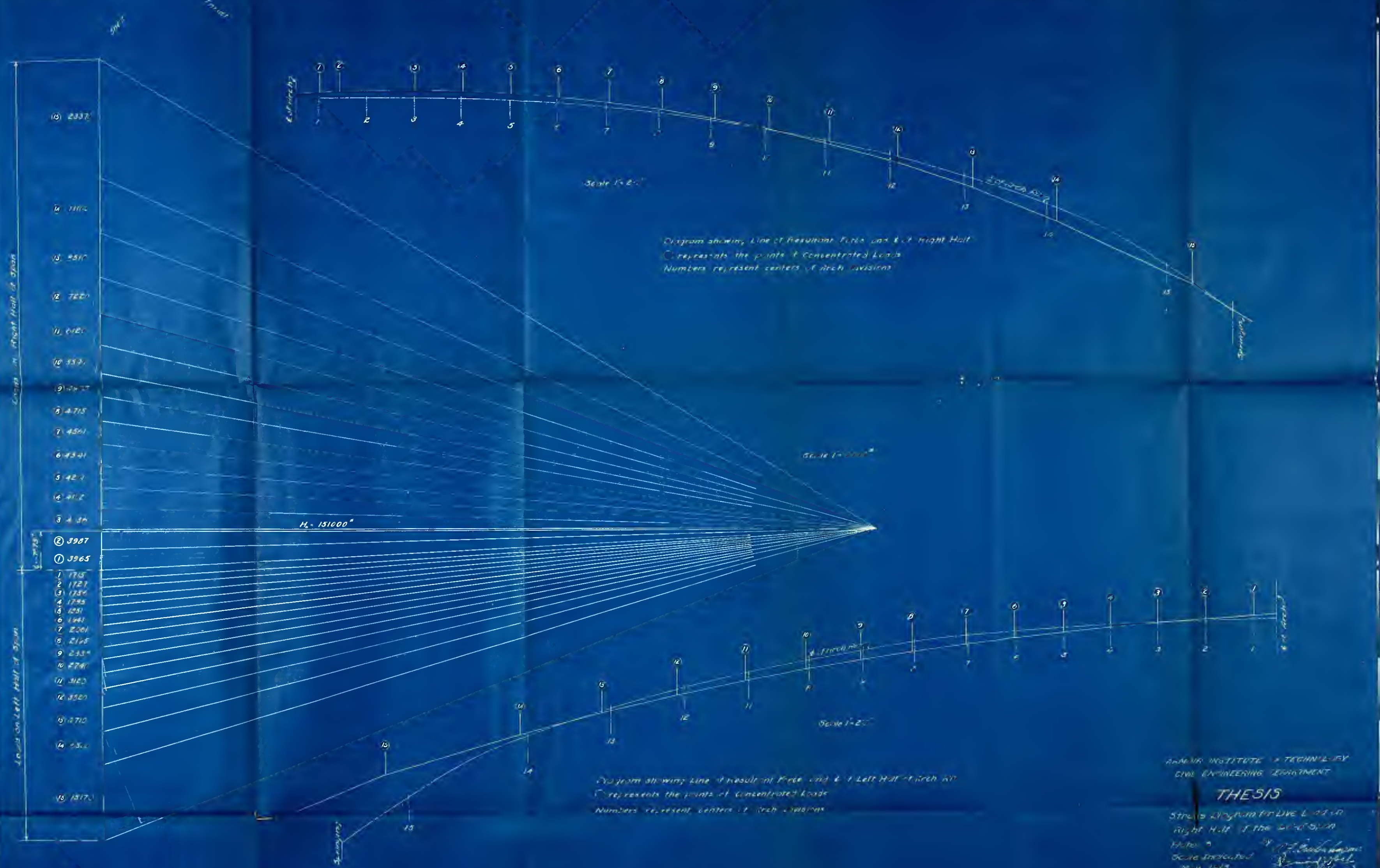


Diagram showing Line of Resultant Force and ℄ of Right Half
● represents the points of Concentrated Loads
Numbers represent centers of Arch Divisions

Diagram showing Line of Resultant Force and ℄ of Left Half of Arch Rib
○ represents the points of Concentrated Loads
Numbers represent centers of Arch Divisions

ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

THESIS

Stress Diagram for Live Load on
Right Half of the Left Span

Plate 9
Scale Indicated
May 1913

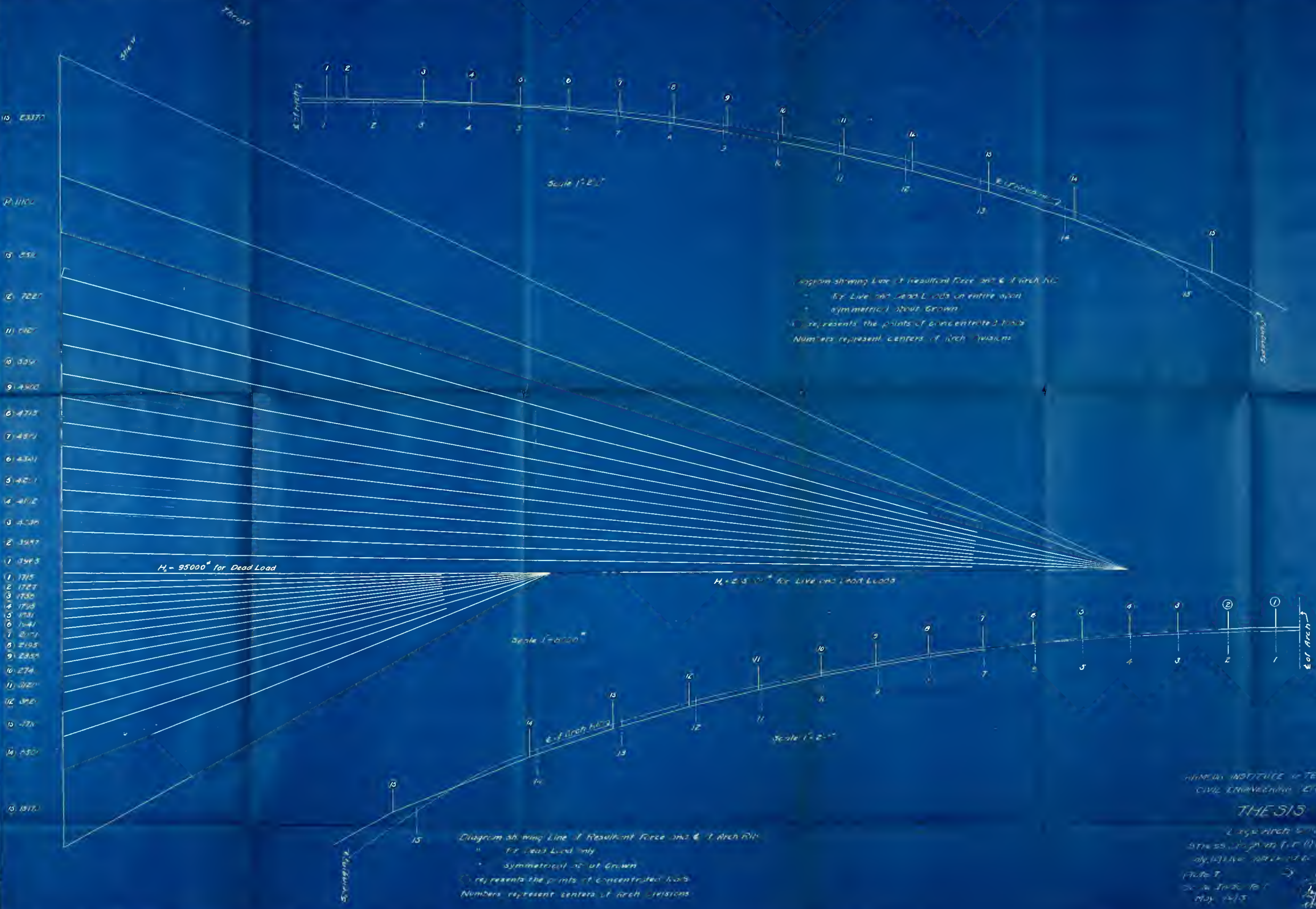


Diagram showing Line of Resultant Force and ℄ of Arch Rib
" for Live and Dead Loads on entire span
" symmetrical about Crown
○ represents the points of concentrated loads
Numbers represent centers of Arch Divisions

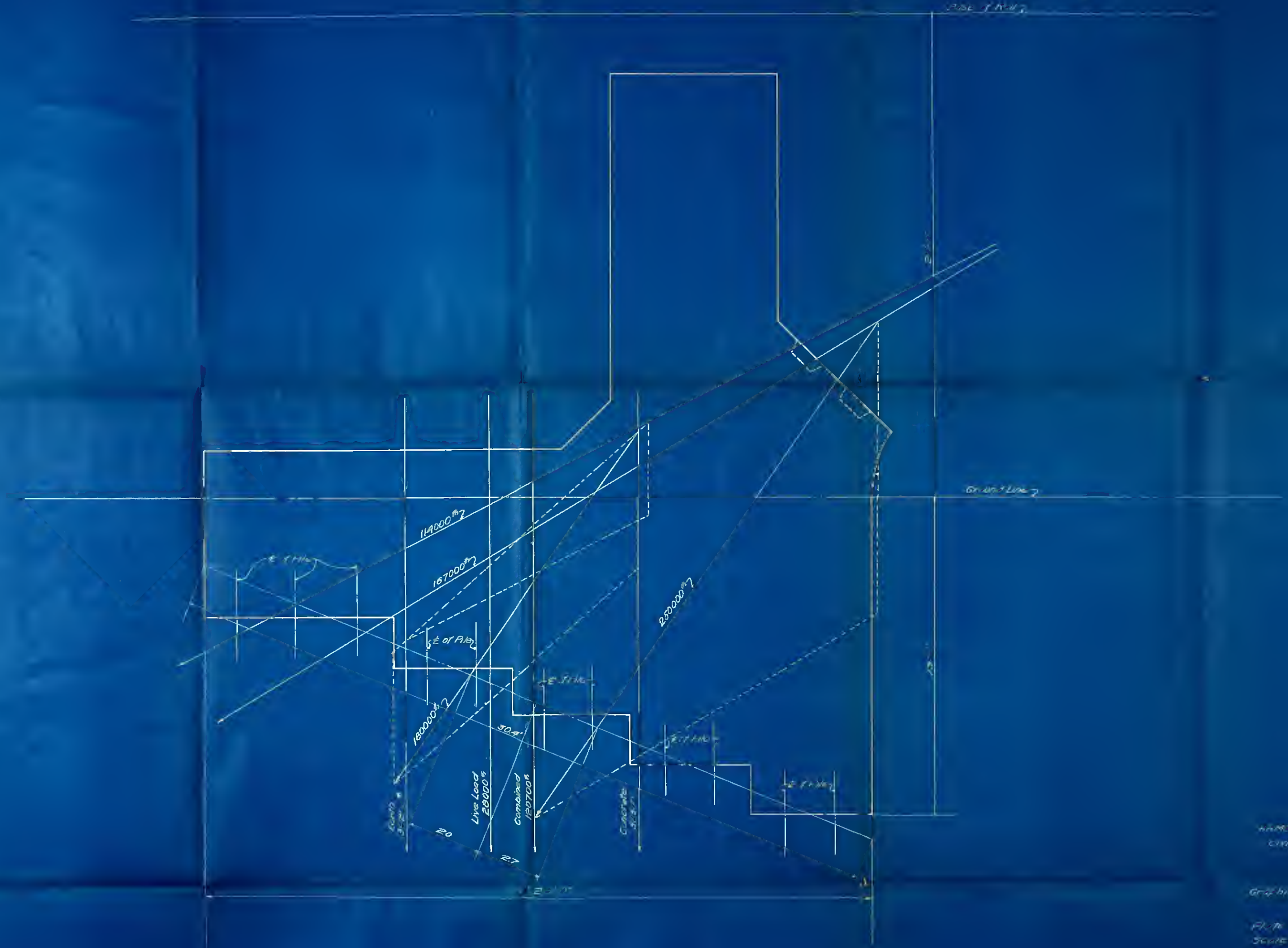
Diagram showing Line of Resultant Force and ℄ of Arch Rib
" for Dead Load only
" symmetrical about Crown
○ represents the points of concentrated loads
Numbers represent centers of Arch Divisions

ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

THESIS

Large Arch 500' Span
Stress Diagram for (1) Dead Load
only, (2) Live and Dead Load on entire span

Plate 7
Scale Indicated
May 1913



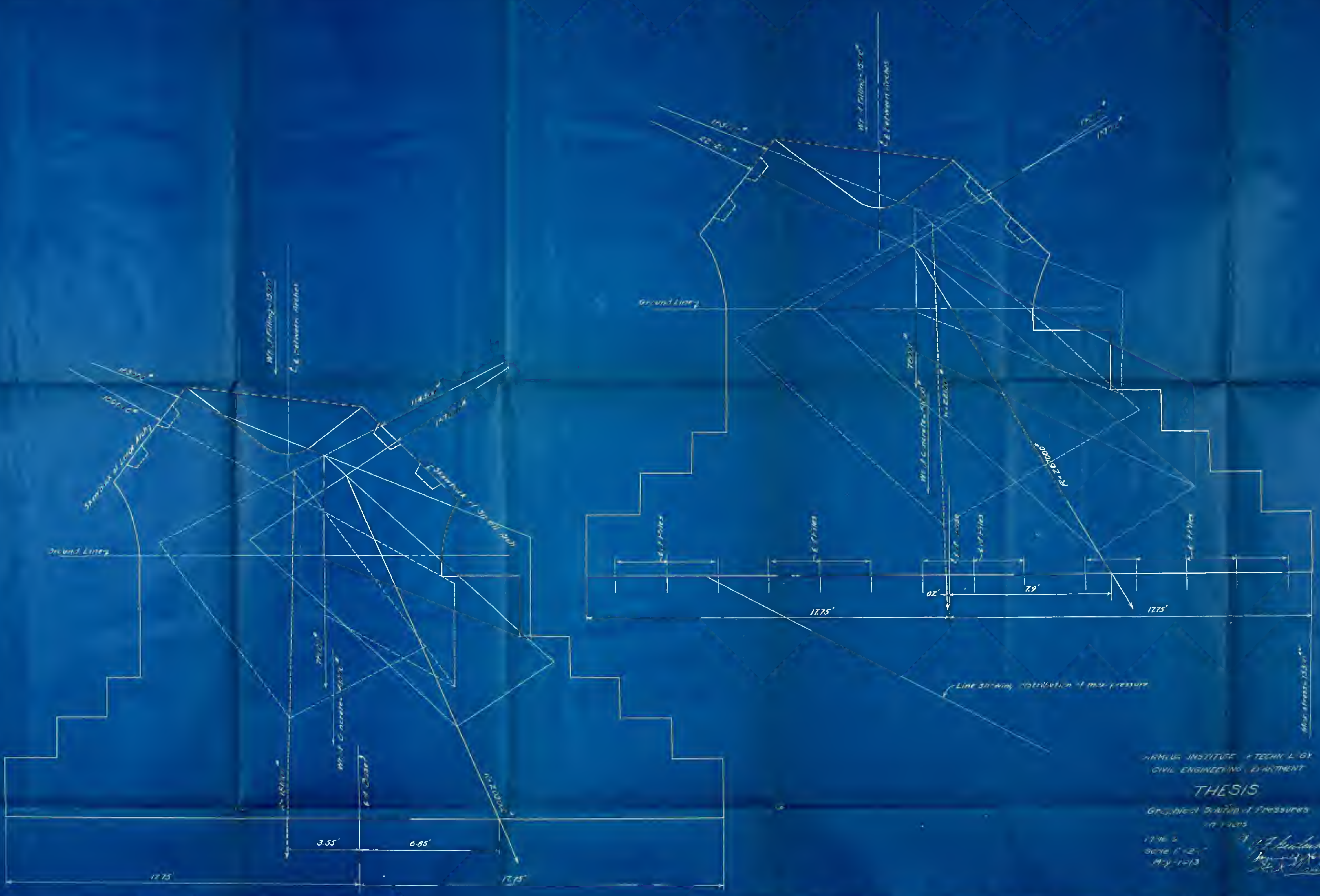
ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

THESIS

Graphical Solution of Pressures on the Abutments.

Plate
Scale 1"=2'-0"
May 1915

by

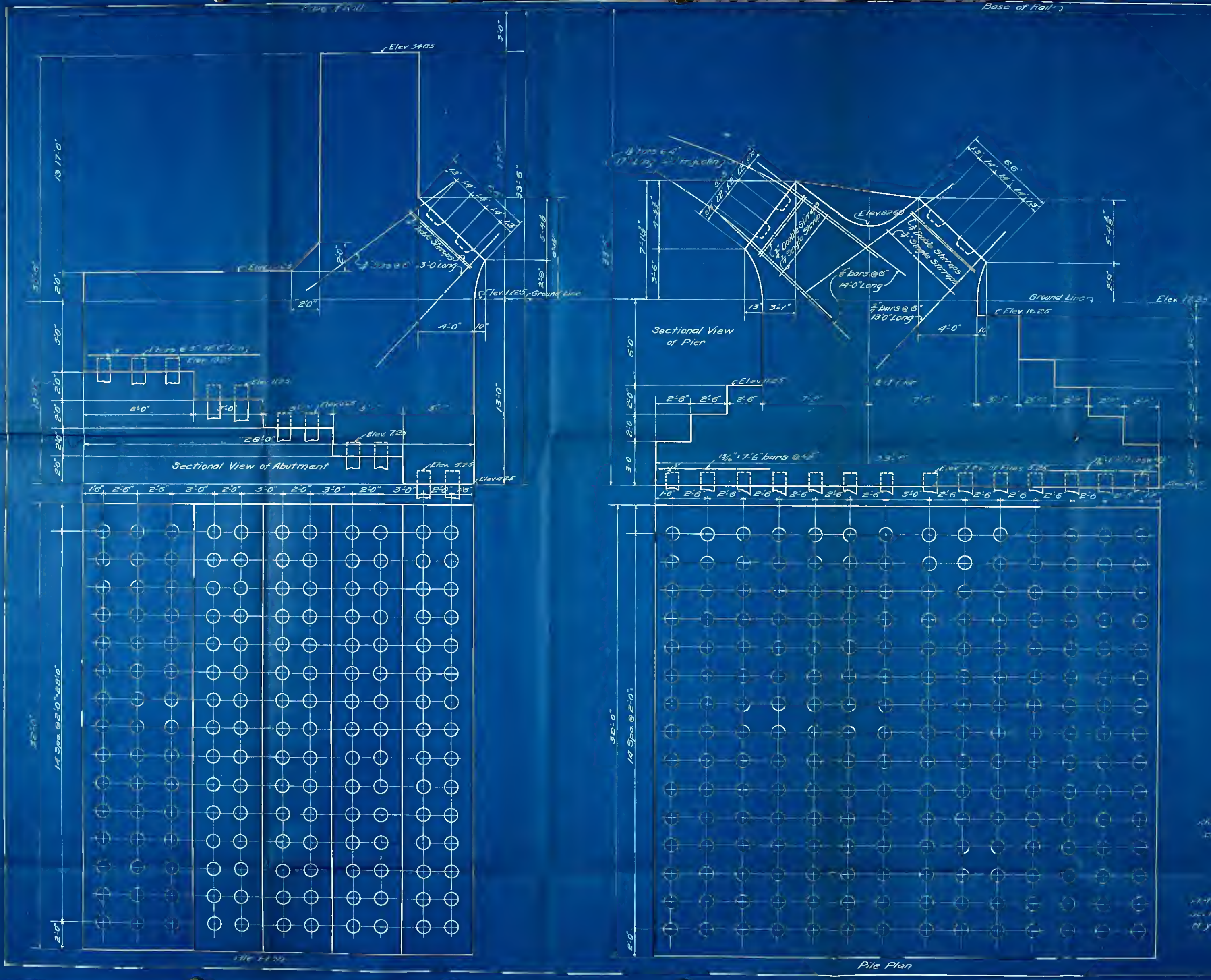


ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

THESIS

Graphical Solution of Pressures
in Piers

Plate 5
Scale 1"=2'-0"
May-1-13



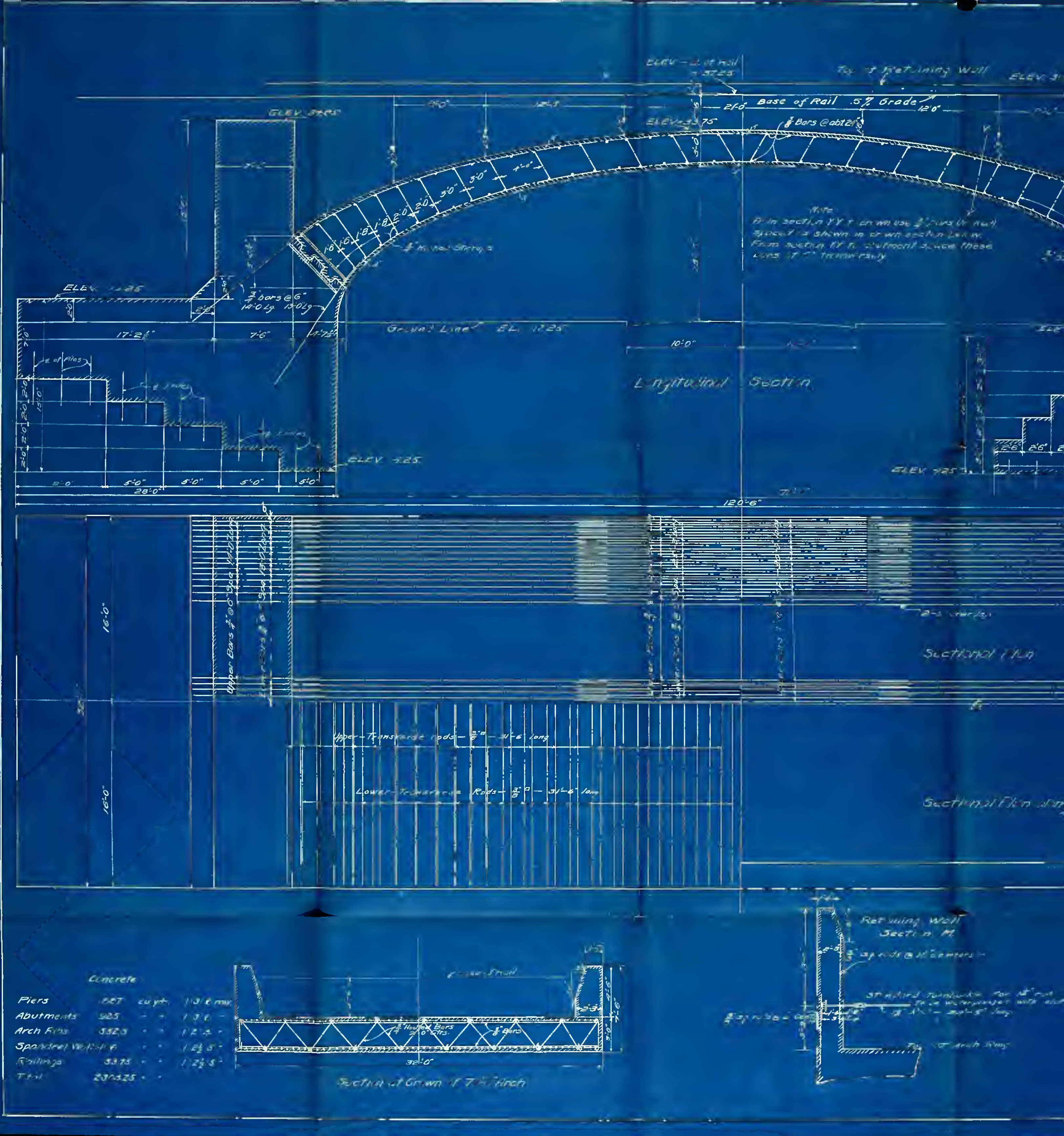
ARMOUR INSTITUTE of TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

THESIS

Piers and Abutment

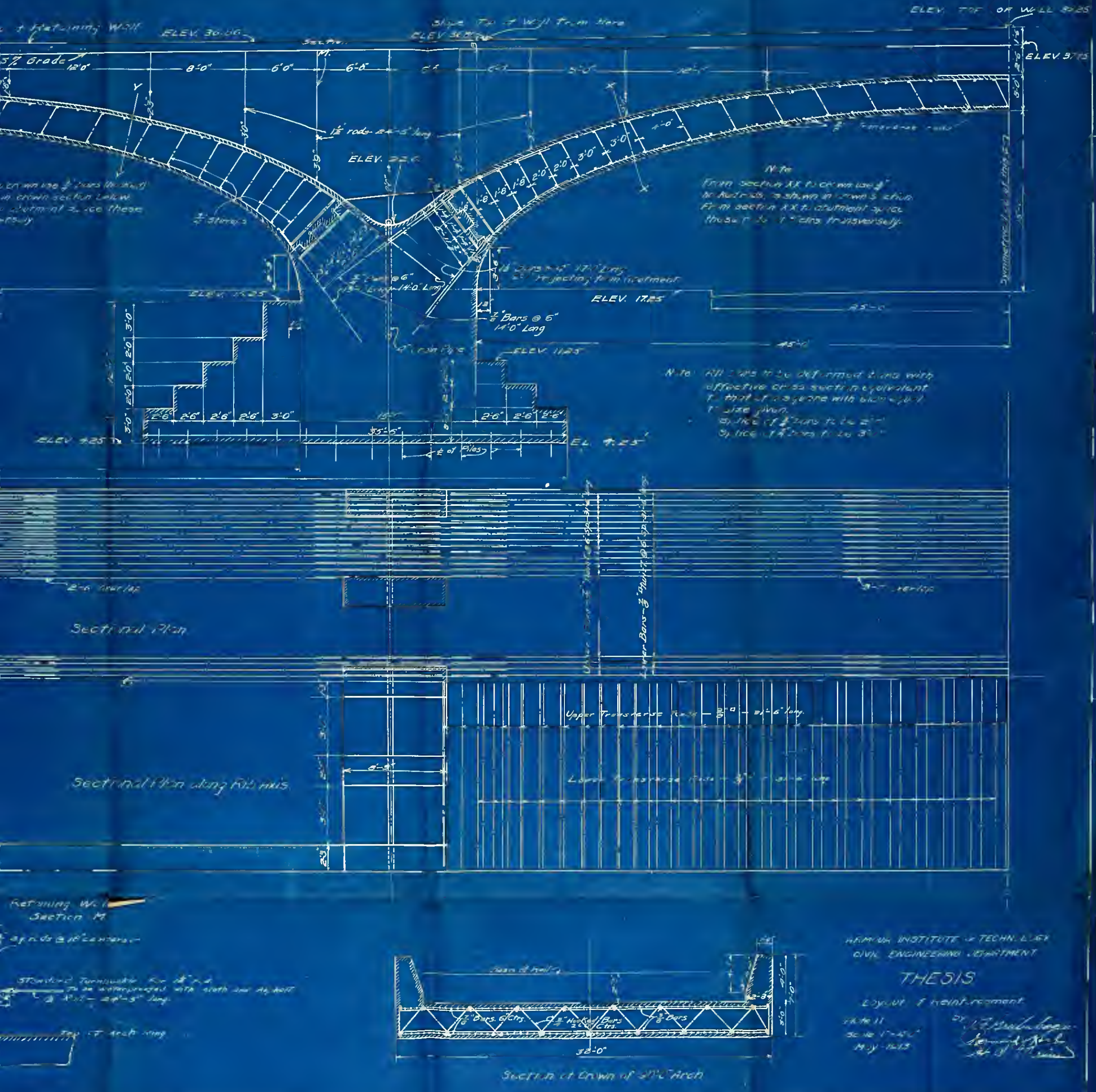
Plate I
Scale 1"=3'
May 1913

by



Note
[illegible] in section YY [illegible] shown in crown section below. [illegible]

Concrete			
Piers	687	cu. yds.	1:3:6 mix
Abutments	925	" "	1:3:6 "
Arch Ribs	332.5	" "	1:2:4 "
Spandrel Walls	[illegible]6	" "	1:2½:5 "
Railings	33.75	" "	1:2½:5 "
Total	2314.25	" "	



Note
From section XX to crown use ⅜" twisted rods, as shown in crown section. From section XX to abutment space these rods at 1" ctrs transversely.

Note: All bars to be deformed bars with effective cross section equivalent to that of a square with side equal to size given.
Splice of ¾" bars to be 2'-"
Splice of ⅞" bars to be 3'-"

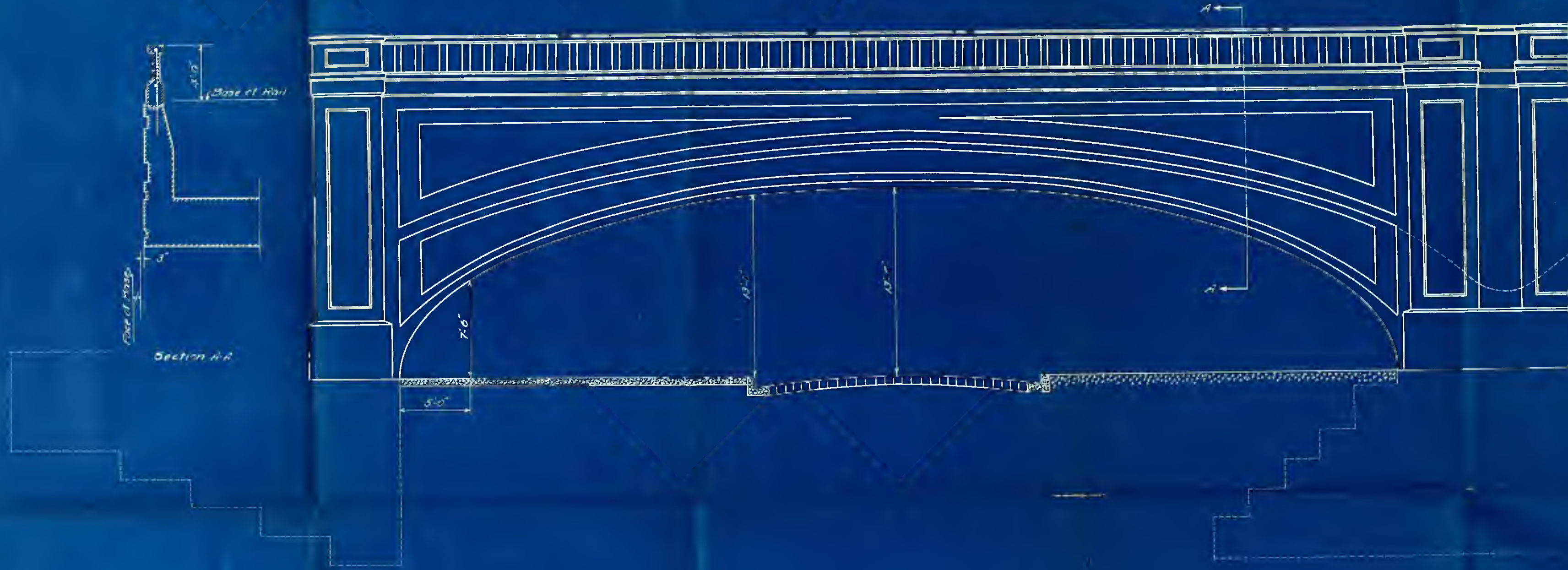
ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

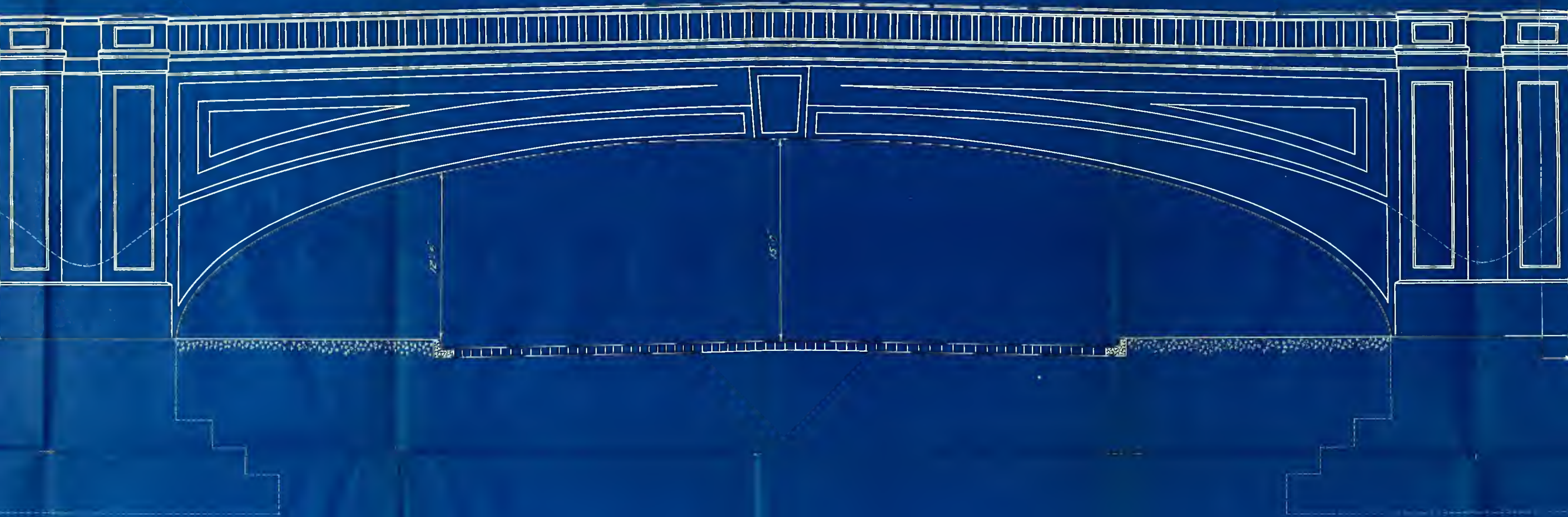
THESIS

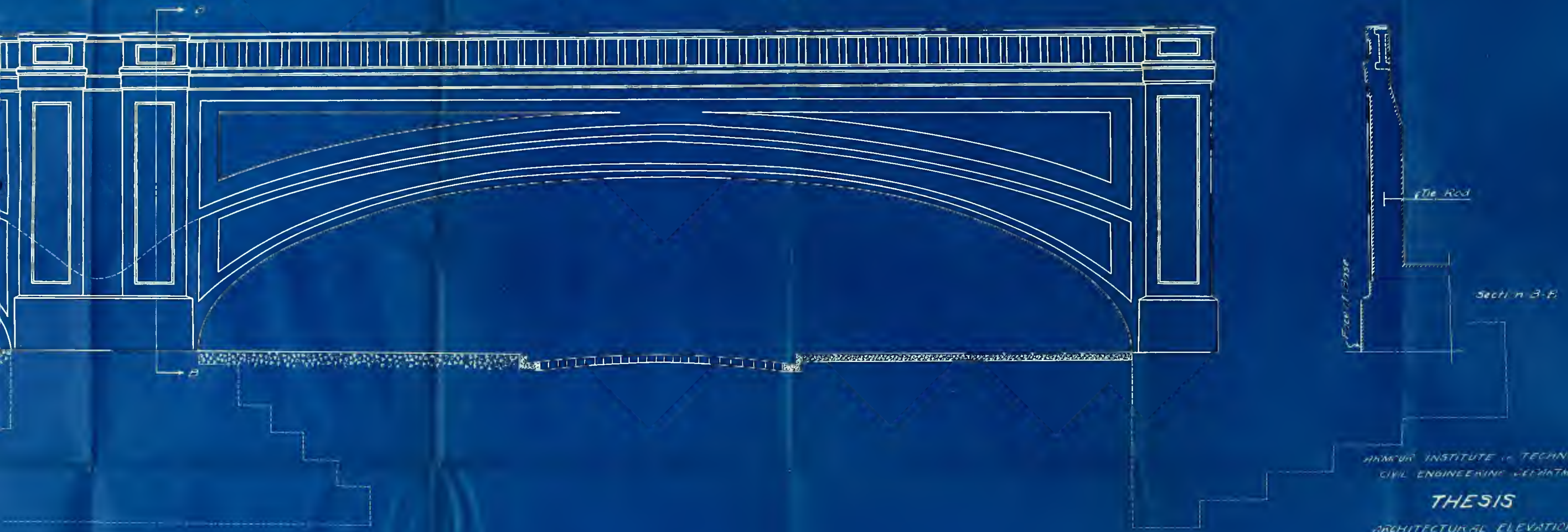
Layout of Reinforcement

Plate 11
Scale 1"=4'0"
May 1913

By

3'-0"
Base of Rail
3"
Face of Post
Section A-A
A
A
7'-6"
5'-0"
13'-
13'-


12'6"
15'0"



ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT

THESIS

ARCHITECTURAL ELEVATION
OF
SUBWAY

Plate 12
Scale 1"=4'0"
May 1913

By